GRAND RIVER BASIN
WATER MANAGEMENT STUDY
TECHNICAL REPORT SERIES

GROUND-WATER RESOURCES IN THE GRAND RIVER BASIN

TECHNICAL REPORT No. 10

Ontario

GRAND RIVER IMPLEMENTATION COMMITTEE

GRAND RIVER BASIN WATER MANAGEMENT STUDY

GRAND RIVER STUDY TEAM

TECHNICAL REPORT SERIES

REPORT # 10

GROUND-WATER RESOURCES IN THE GRAND RIVER BASIN

PREPARED FOR THE GRAND RIVER IMPLEMENTATION COMMITTEE BY:

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Ministry of the Environment

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FOREWORD

This report is one in a series of technical documents prepared for the Grand River Basin Water Management Study. The project described herein was undertaken by the Grand River Hydrology Sub-Committee at the request of the Grand River Implementation Committee.

The material contained in this report is primarily technical support information and in itself does not necessarily constitute policy or management practices. Interpretation and evaluation of the data and findings, in most cases, cannot be based solely on this report but should be analysed in light of other reports produced within the comprehensive framework of the overall study. Questions with respect to the contents of this report should be directed to the Co-ordinator of the Grand River Basin Water Management Study, Water Resources Branch, Ministry of the Environment, 135 St. Clair Avenue West, Toronto.

ACKNOWLEDGEMENTS

The ground-water inventory study was assisted by numerous people throughout the two-year duration. Of these participants, the assistance of W. Leipciger throughout the study was invaluable and R. Dicken contributed significantly to the evaluation of supplies for municipal purposes. Able assistance was provided by students during the study and special appreciation is accorded to residents of the basin who co-operated during phases of the study involving test drilling, water sampling, and the installation of observation wells.

Metric Conversion Factors

1 foot	=	0.305 metres
1 mile	=	1.609 kilometres
1 square mile	=	2.590 square kilometres
1 gallon	=	4.546 litres
1 gallon/minute	=	0.0758 litres/second
1 gallon/day	=	5.262×10^{-5} litres/second
1 gallon/minute/foot	=	0.249 litres/second/metre

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SUMMARY

The purpose of this study was to determine the availability of ground water throughout the Grand River basin, with special emphasis on the availability of ground-water supplies for municipal uses. In 1977, the cumulative municipal ground-water demand within the basin was over 40 million gallons per day (mgd). By the year 2031, this demand is expected to increase to over 100 mgd. It is significant to note that more than 90 percent of the municipal ground-water demands in the basin originate from three population centres - Guelph, Kitchener-Waterloo and Cambridge. This disproportionate water demand in the central part of the basin has strained known ground-water resources in the area, particularly in Kitchener-Waterloo where short-term water shortages have been experienced in the past. Thus, an assessment of the potential for additional municipal ground-water development, particularly in the central portion of the basin, is of major importance.

The Grand River basin encompasses an area of 2614 square miles, with an approximate length of 118 miles and an average width of 22 miles. Elevations in the basin vary from a high of approximately 1750 feet above mean sea level near Dundalk, to a low of about 570 feet near Port Maitland at Lake Erie. The physiography of the basin is fairly representative of physiography in other parts of southern Ontario, and can be separated into three broad classifications. The northwestern and northern parts of the basin consist mainly of rolling till plains of varying relief, the central portion consists mainly of moderate to high-relief landforms associated with sand and/or gravel deposits, and the southern part of the basin consists mainly of a clay plain with relatively low relief.

The principal tributaries of the Grand are the Nith, the Speed and Eramosa, and the Conestogo rivers.

Bedrock in the basin includes ten geologic formations, but most of the basin is underlain by only three of these: the Salina Formation, which underlies the western half and southern portion of the basin, and consists of dolomite, limestone, shale and evaporites, and; the Guelph and Amabel-Lockport formations, which underlie the eastern half and northern portion of the basin, and consist of dolomites.

Ground-water supplies in the basin originate from permeable sand and/or gravel deposits within the overburden, or from bedrock. The highest ground-water yields from overburden are found in the central parts of the basin, particularly around Kitchener-Waterloo and Cambridge, and in areas of localized sand and gravel deposits in the northern portions of the basin and in buried bedrock valleys such as the one between Elora and Fergus. The highest ground-water yields from bedrock, in excess of 200 gpm, are found in the central, northwestern and northern parts of the basin, generally corresponding to areas of permeable limestone and dolomite bedrock. The Guelph-Amabel complex in particular is recognized as one of the most productive aquifer complexes in Ontario.

The overburden ground-water quality is similar throughout the basin, with minor variations reflecting localized conditions, and is generally superior to bedrock ground water with respect to criteria for potable supplies. The quality of ground water from bedrock varies with location in the basin and generally with depth of well penetration into bedrock. Wells obtaining water from the Salina Formation generally contain higher values for sulphates, iron and hardness than do the wells developed in the Guelph, Amabel and Lockport formations. High sulphate waters from bedrock are most common throughout the southwestern and southern parts of the basin.

The municipal water supply and demand situations of 23 communities in the Grand River basin were evaluated in this report. Four of these communities (Burford, Drayton, Grand Valley and Salem) did not have municipal water-supply systems in 1979, but will likely be using ground water for municipal supplies in the future. The existing supplies of 14 communities were found to be adequate to meet the respective water demands in the year 2031, and the remaining 5 communities (Cambridge, Elora, Fergus, Guelph and Kitchener- Waterloo) appear to require additional supplies to meet the projected demands in 2031. The potential for future ground-water development by most communities appears to be good.

Since the Regional Municipality of Waterloo is in the process of conducting an extensive study of ground-water resources in the Cambridge area (results will be available late in 1980), an evaluation of future supplies for Cambridge was not carried out in this study.

An investigation of the water supply and demand situation for Kitchener-Waterloo included an evaluation of proposed water-supply development projects. These projects include induced infiltration developments at four locations along the Grand River, artificial ground-water recharge developments near Roseville and Mannheim, and natural ground-water development north of Roseville. An evaluation of design criteria, costs and projected yields was carried out for each proposed project.

This report also examined certain ground-water management issues and identified the most common problems associated with ground-water development. These include past well interference and contamination problems, poor natural water quality, flowing and dry wells, and the susceptibility of ground water to contamination in some areas. An assessment of the long-term ground-water potential in the basin indicated that supplies may exceed the 2031 estimated municipal demands by about 37 per cent. However, only site-specific comparisons of supply and demand can be meaningful in determining the adequacy of ground water to meet each of the individual municipal demands in the basin.

GROUND-WATER RESOURCES IN THE GRAND RIVER BASIN

1. INTRODUCTION

1.1 Purpose and Scope

The general megalopolis area of Kitchener-Waterloo, Cambridge and Guelph in the central part of the basin is the largest in the Province that depends almost exclusively on ground water for municipal supplies. In addition to this large centre of urban activity, there are 16 other communities in the basin that depend wholly on ground water for their domestic, commercial and industrial needs. In 1977, the total average-day consumption of ground water by all the communal systems was over 40 mgd, and the demand by the year 2031 is expected to be almost triple. Spread evenly over the basin, the future demand may not seem particularly striking; however, the demand for this water is concentrated in the megalopolis area in the central part of the basin where the projected average-day demand in 2031 will be about 95 mgd, or approximately 92% of the ground water needed for municipal supplies in the whole basin.

Because of the continuous increase in demand for supplies in the Kitchener-Waterloo area in recent years, the Regional Municipality of Waterloo has concentrated efforts to seek new sources of ground water in the area of the twin cities. The radius of search has increased outward from the cities into adjacent rural areas and consequently conflicts with rural water needs have arisen. With this in mind, the present study was designed to determine the availability of ground water throughout the basin, with special emphasis on the availability for municipal needs, so that future water use conflicts might be minimized and the short and long-term options for large-scale water supplies in the basin might be formulated.

The inventory of the available ground-water resources was divided into two general areas of study:

1. a general inventory of ground-water quantity and quality throughout the basin; and
2. an inventory of ground water available for future development by each of the municipalities in the basin that either have existing municipal systems utilizing ground water, or are likely to use ground water in the future.

The general inventory covered the whole basin and dealt with defining areas that have the potential to yield 50 gallons per minute (gpm) or more from single wells. Because adequate quantities of ground water for single household purposes are generally not a problem in most parts of the basin, the 50 gpm cut-off for the investigation reflects the basic purpose of the study to locate large quantities of ground water needed typically for municipal or industrial uses. The general inventory also included determining the range of water quality in the basin, estimating natural recharge to ground water, mapping aquifers that may be susceptible to contamination from surface sources, and determining general problem areas associated with flowing wells, poor natural water quality, insufficient ground- water supplies to meet domestic requirements, and locating sites of potential ground-water contamination by activities such as landfill operations.

The inventory for municipal supplies involved in-depth analysis of the potential for future ground-water development by each municipality in the basin that has or may be expecting to utilize ground water in the future. The study for Kitchener-Waterloo received special attention because of the large demand and existing shortages of supplies during summer peak-demand periods. The City of Cambridge was excluded from an in-depth analysis because a detailed exploration program was being carried out concurrently with this study in 1978 and 1979.

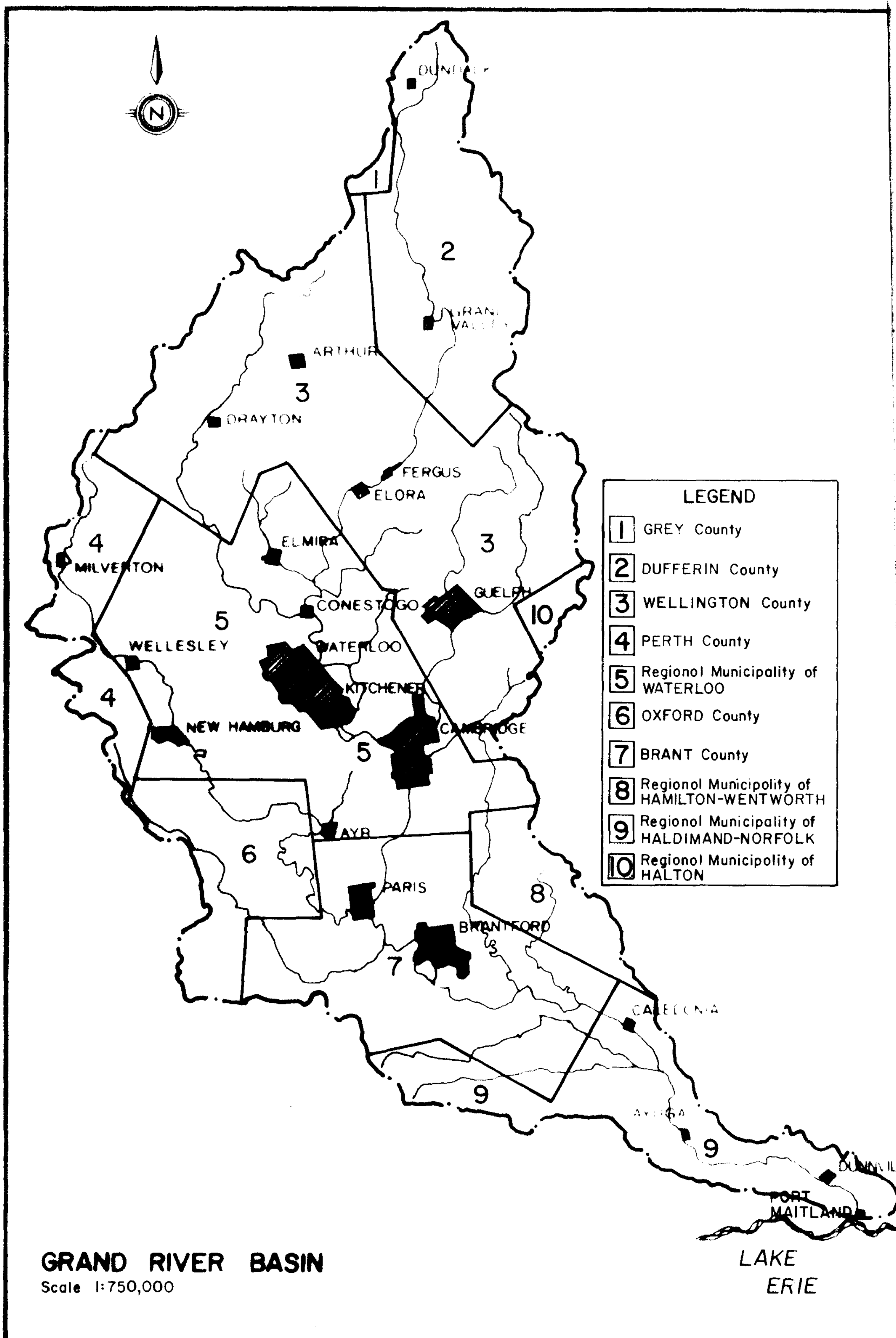
In addition to determining the inventory of the natural occurrence of ground water for municipal supplies for Kitchener-Waterloo, the preliminary feasibility of water-supply options such as artificial recharge and induced infiltration (already being used by Kitchener-Waterloo) were also studied. All options are provided with cost estimates.

Field work during the study consisted of installing 5 observation wells to provide background water-level data in a number of significant hydrologic conditions in the basin, obtaining water quality samples from major aquifers, and test drilling in 1979 to provide site-specific information on hydrogeology in areas considered to have a significant potential for development of future ground-water supplies for municipal uses. Additional test drilling in surficial deposits, together with geophysical exploration, was carried out to determine the potential for artificial recharge and induced infiltration at the proposed West Montrose reservoir site.

The basic sources of hydrogeologic data for the study were the approximately 10,000 water well records in the basin on file with the Ministry of the Environment up to the end of 1978.

1.2 Location

The Grand River basin encompasses an area of 2614 square miles in southcentral Ontario, and drains into Lake Erie at Port Maitland (Figure 1.1). The basin is approximately 118 miles long and has an average width of 22 miles. Major municipalities include Kitchener-Waterloo, Cambridge, and Guelph in the central part of the basin, and Brantford in the southern part. The basin contains practically all of the Counties of Wellington and Brant, and the Regional Municipality of Waterloo, and parts of the Counties of Grey, Dufferin, Perth and Oxford, and the Regional Municipalities of Hamilton-Wentworth, Halton and Haldimand-Norfolk.



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1.3 Physiography and Drainage

The basin contains a wide variety of landforms that consequently results in varied drainage patterns. All of the landforms are related to the last glaciation of approximately 12,000 years ago, and to post-glacial erosion subsequent to the withdrawal of the ice from this part of Ontario. The landforms include the normal range of features found in southern Ontario where the overburden is relatively thick over Paleozoic bedrock. Notable large-scale features include the large, low-relief clay plain in the southern part of the basin, till plains of varying relief in the north-western and northern parts of the basin, and areas of moderate to high relief associated with deposits of sand and/or gravel notably in the area around Guelph and between Kitchener-Waterloo and Brantford.

On a smaller scale, notable physiographic features include the high-relief Baden (sand) Hills just east of Baden and the Elora Gorge, which is sculptured into limestone by the Grand River at Elora.

Elevations in the basin vary from a high of approximately 1750 feet above mean sea level near Dundalk to a low of about 570 feet at Lake Erie.

There are three large tributaries to the Grand River: the Nith River, the Speed River (including the Eramosa River) and the Conestogo River. The Nith River drainage basin has the largest area (432 mi^2) and the river itself is approximately 98 miles long. The Conestogo River basin has an area of 317 mi^2, with a river length of 51 miles, and the Speed River basin has an area of 301 mi^2 and a river length of 37 miles.

Other notable, but smaller tributaries are Fairchild Creek, Whiteman's Creek, McKenzie Creek, Boston Creek and Big Creek.

Most of the drainage patterns in the basin are related to land physiography and surficial materials. In general, the densest system of streams occurs in hummocky areas of the basin covered by till and clay where the infiltration of precipitation into the ground is generally low. A notable area containing many streams is located on the rolling clay plain area east and south of Brantford. The least number of streams usually occurs in areas of surficial sands and/or gravels because the rate of infiltration of precipitation is generally high. An example of this can be found in the sand and gravel areas between Paris and Kitchener-Waterloo where there are very few streams.

2 GEOLOGY

2.1 Introduction

An understanding of bedrock and overburden geology is fundamental in determining the occurrence and distribution of high-yield water-bearing zones in the watershed. Consequently, this chapter deals with descriptions of basic geologic formations that generally have a direct bearing on ground-water conditions in the basin. The bedrock formations that subcrop within the basin are identified and discussed in relation to their stratigraphy, lithology, thickness and lateral extent. Variations in bedrock topography are also discussed. A brief history of glacial activities during Late Wisconsinan times is outlined as background for discussions of lithology, lateral distribution and the thickness of surficial and total overburden deposits in the basin.

Bedrock geology in the eastern portions of the basin was compiled from 1:50,000-scale maps published by the Ontario Division of Mines (ODM). To complete the bedrock geology for the remainder of the basin, information was taken from the 1:250,000-scale map by Sandford (1969). Bedrock terminology is generally consistent with that used by the Geological Survey of Canada (1970).

Bedrock topography was compiled primarily from 1:50,000-scale maps published by the ODM, which covered most of the basin. Additional bedrock topography information was obtained from ground-water survey report for the Town of Fergus (Andrijiw, 1976) and from test drilling carried out by the MOE during the summer of 1979.

Information about overburden geology was derived from ODM reports and from publications by Cowan (1972, 1975, 1976) and Karrow (1961, 1968, 1971, 1974), supplemented by field investigations and test drilling in 1979. Terminology used in the discussion of glacial deposits is after Karrow (1974) and Cowan (1975).

Information on the thickness of overburden deposits has been obtained from published geologic maps and reports. The bedrock valleys in the vicinity of Elora-Fergus and northeast of Guelph have extremely limited widths, making it difficult to represent the thickness of the overburden at the scale of 1:250,000 as it appears on Figure 2.3.

2.2 Bedrock Geology

2.2.1 Stratigraphy

The bedrock underlying the Grand River basin consists of marine sediments deposited during the Silurian and Devonian periods of the Paleozoic Era, 370 to 345 million years ago. The bedrock formations that subcrop within the basin consist of the Upper Ordovician Queenston Formation, the Lower Silurian Cataract Group, Middle and Upper Silurian Amabel-Lockport, Guelph, Salina and Bass Islands formations, and the Lower to Middle Devonian Oriskany, Bois Blanc and Amherstburg-Onondaga formations. Their distribution, as mapped by Sandford (1969), is indicated on Figure 2.1.

The Queenston Formation subcrops as the basal unit in the Dundas bedrock valley near Copetown west of Hamilton. It consists of red shale and mudstone with minor interbeds of silty limestone. The Cataract Group overlies the Queenston Formation and also subcrops in the Dundas valley. The Cataract Group consists of the Whirlpool, Manitoulin, and Cabot Head formations. The Whirlpool Formation is a white and grey sandstone, the Manitoulin Formation consists of a brown and grey dolomite, and the Cabot Head Formation is made up of greenish grey and red, silty shale.

The Amabel-Lockport formations subcrop in the extreme eastern portions of the basin. The Lockport Formation goes through a facies change at Waterdown and extends north to the Bruce Penninsula as the Amabel Formation. The formation consists of fine to medium, crystalline buff and grey, to dark brown and black dolomite, with minor shale.

The Guelph Formation subcrops to the east and consists of buff to grey, finely crystalline to granular dolomite, with major bioherm reef development. The combined thickness of the Guelph and Amabel-Lockport formations in the basin is approximately 300 feet.

The Salina Formation subcrops west of the Guelph dolomites. It forms the bedrock through much of the westcentral portion of the basin and consists of grey-brown dolomite, grey and red shale, minor limestone and evaporite deposits composed of salts, anhydrite and gypsum. The formation is approximately 300 feet thick in the Regional Municipality of Haldimand-Norfolk, increasing in thickness towards the northwest.

The Bass Island (Bertie) Formation subcrops west of the Salina and consists of finely crystalline brown dolomite with minor, thinly-bedded shaley dolomite. The formation is about 400 feet thick in the basin.

The Lower Devonian Oriskany Formation subcrops in a limited area in Oneida and North Cayuga townships. The formation consists of light grey to white, medium - grained sandstone, which rarely exceeds 15 to 20 feet in thickness.

The Bois Blanc Formation subcrops in a narrow band west of the Bass Island Formation. It consists of bluish-grey, finely crystalline silty limestone and sandstone, 15 to 40 feet thick in the Niagara Peninsula and increases in thickness to the northwest.

The Middle Devonian Amherstburg - Onondaga formations subcrop in the extreme western portions of the basin. The Amherstburg Formation consists of brown dolomite, as thick as 300 feet, which thins eastward and grades into the Onondaga Formation in the vicinity of Waterford. The Onondaga Formation is estimated to be approximately 70 feet thick within the basin and consists of grey, coarse-grained limestone and dark grey, fine-grained cherty limestone.

2.2.2 Topography

The bedrock surface in the Grand River basin is irregular, with surface elevations ranging from less than 450 feet above sea level near the mouth of the Grand River, to a maximum of about 1650 feet in the headwater regions in the northern tip of the basin. Bedrock elevation trends throughout most of the basin indicate a complex system of channels produced by erosion of the bedrock surface by (presumably) preglacial drainage. The bedrock valleys indicate a drainage pattern trending generally from northwest to southeast. The most prominent of these valleys are the Dundas valley in southern Wellesley Township, the Elora-Fergus valley north of Kitchener-Waterloo, and a valley trending southwest from Brisbane to the City of Guelph. The Onondaga Escarpment is a relatively low-relief feature situated south of the Grand River and extends from about Hagersville eastward, following approximately the 600-foot bedrock topographic contour.

2.3 Overburden Geology

2.3.1 Surficial Deposits

The surficial materials in the Grand River basin were deposited as the result of a series of glacial advances and retreats during the Late Wisconsinan glaciation. In order to understand the variety of surficial deposits in the basin, a brief summary of the glacial activity during this period is necessary.

TABLE 2.1 Sequence of Glacial Activities in the Basin

Stage	Substage	*Stadials and Interstadials	Years (B.P.)
		*Port Huron	13,000
Wisconsin	Late	Mackinaw	14,000
	Wisconsin	*Port Bruce	15,000
		Erie	17,000
		*Nissouri	24,000

Advance of glacial ice during the Nissouri Stadial, approximately 24,000 years ago, led to the inundation of the basin by ice. The ice sheet covered all of southern Ontario and extended into southern Ohio, depositing a till unit identified as Catfish Creek Till.

Retreat of the glacier in southern Ontario during the latter stages of the Nissouri Stadial started with the thinning and splitting of the ice sheet along a line from the Kitchener-Waterloo area to northeast of Orangeville. Glacial activity in the Grand River basin for the remainder of the Late Wisconsinan involved three ice lobes, the Georgian Bay Lobe from the north, the Erie-Ontario Lobe from the east and southeast, and the Huron Lobe from the west and southwest.

Retreat of the glacial ice continued until the Georgian Bay and Huron lobes had receded to positions outside the drainage basin. During this period of deglaciation (the Erie Interstadial), meltwaters from the receding ice lobes created extensive glaciofluvial deposits throughout the basin. Ponding of meltwaters resulted in numerous proglacial lakes and the deposition of glaciolacustrine silts and clays. Initial deposition in the area of the present Waterloo and Orangeville moraine complexes is thought to have taken place at this time.

During the Port Bruce Stadial, each advancing ice lobe deposited extensive till sheets composed largely of reworked glaciolacustrine deposits. The major advance of the Georgian Bay Lobe resulted in the deposition of the Tavistock Till, which forms the surficial materials throughout much of the northern and northwestern portions of the basin. Two other tills, deposited as a result of minor fluctuations of the Georgian Bay Lobe, form the surficial materials in the western portions of the basin and have been identified as Mornington and Elma tills. The advancing Erie-Ontario Lobe deposited the Port Stanley Till and created the Guelph drumlin field, while the Huron Lobe deposited a till sheet first identified as Zorra Till by Cowan (1972) and later renamed as the Tavistock Till by Karrow (1974). A minor fluctuation of the Huron Lobe during the latter stages of the Port Bruce Stadial resulted in the deposition of the Stratford Till, which forms the surficial material in the vicinity of Nithburg.

Recession of the three ice lobes in the late Port Bruce Stadial resulted in the deposition of extensive kame, outwash and glaciolacustrine materials throughout much of the central portions of the basin. These materials compose the bulk of the Waterloo and Elmira moraine complexes and exist throughout the Regional Municipality of Waterloo. In addition, the bulk of the Orangeville moraine complex, the southern extent of which exists in Eramosa, Erin and East Garafaxa townships, was created at this time. Meltwaters from the receding Erie-Ontario and Georgian Bay lobes created a complex of outwash channels that form a general pattern of winding, concentric arcs trending in a southwesterly direction across the area enclosed by Guelph, Fergus and Kitchener-Waterloo. Many modern streams, such as the Grand River and Swan Creek, occupy portions of these channels. Successive positions of the receding Erie-Ontario Lobe are well marked by a series of terminal moraine deposits, with associated kame and outwash materials, west of the town of Burford. By the time of the Mackinaw Interstadial, the entire basin area was free of ice.

Subsequent glaciation during the Port Huron Stadial saw only the Erie-Ontario Lobe re-enter the basin, depositing the Wentworth Till. The limit of the Port Huron advance is marked by the position of the Paris Moraine. Recession of the Port Huron ice sheet resulted in the deposition of ice-contact deposits in association with Wentworth Till to further build up the Paris and Galt terminal moraines. Meltwater flowed in a southwesterly direction, creating outwash channels that extend from Guelph to Burford. Retreat of the Erie-Ontario Lobe from the basin marked the end of glaciation in the area.

The recession of the Port Huron ice sheet was accompanied by the creation of glacial Lake Wittlesey, which may have existed until the Port Huron ice had retreated almost to the Niagara Escarpment. A series of glacial lakes continued to inundate the basin south of the City of Brantford until about 12,000 years ago. The deep-water lacustrine sediments throughout the southern portions of the basin are thought to have been deposited by glacial Lake Warren II, formed when lake levels rose with advance of the Halton ice sheet east of the basin. The present drainage system was established approximately 12,000 years ago.

The general surficial deposits present in the basin are depicted on Figure 2.2. There is a high degree of lateral variability in these materials, so that surficial deposits with similar lithologies have been grouped into three categories, with areas where bedrock outcroppings occur, grouped into a fourth. These groupings represent general geology and reflect a cursory distinction between permeable sands and/or gravels, and less permeable tills, silts and clays. Shallow overburden and bedrock areas are also considered to contain permeable surficial materials.

(i) Sand and Gravel

Surficial deposits of sands and/or gravels generally relate to kame and outwash, shallow-water lacustrine sediments, and beach and deltaic materials. Kame and outwash deposits consist of fine to coarse-grained sands and/or gravels in glacial meltwater channels in

contact with, or in front of, a stagnant or retreating ice margin. Kame materials are often deposited with inliers of silt, clay and till, and may be associated with tills in terminal moraine complexes. These materials are variable in composition, lateral extent and thickness. Outwash sediments consist mainly of stratified sands and gravels deposited in meltwater channels emanating from the terminus of an ice sheet. Shallow-water lacustrine deltaic and beach sediments consist primarily of medium to fine sands, and in some cases, reworked outwash gravels and tills. Nearshore sediments consist mainly of fine sands, while surficial sand and gravel deposits predominate throughout much of the central portion of the basin and range from a few feet to over 100 feet in thickness. The most extensive sequences occur throughout most of the R. M. of Waterloo and in Wellington, Brant and Oxford counties.

Within Wellington County, surficial sands and gravels occur in Eramosa, Erin and East Garafraxa townships. They generally compose kame and outwash deposits and minor glaciolacustrine materials in the southern extension of the Orangeville Moraine. The deposits in these townships range from 50 to 100 feet in thickness throughout much of the area and may exceed 150 feet in thickness locally. Numerous outwash channels containing gravels and gravelly sands, frequently overlain by several feet of silt, exist in the area.

The most extensive surficial deposits of sands and gravels in the basin occur throughout most of the R. M. of Waterloo. The materials, for the most part, compose the Waterloo and Elmira moraine complexes and consists of poorly to well-sorted kame sands and gravels, with associated outwash channels containing well-sorted, fine to coarse sand and fine to medium gravel. Thicknesses of surficial sands and gravels throughout this area range generally from 20 to 50 feet, with deposits of up to 125 feet occurring locally in Wilmot, Woolwich and Wellesley townships.

Sands and gravels situated in the vicinity of Guelph, and extending into North and South Dumfries and Brantford townships, are part of the kame and outwash materials deposited in association with the Paris and Galt moraines. These deposits are up to 60 feet thick in the vicinity of Paris.

Within Oxford County, large areas of surficial sands and gravels occur in the vicinity of the towns of Blandford and Gobles, and have been identified by Cowan (1975) as outwash sands. The permeable surficial materials extending northeast, from the Drumbo area, are composed primarily of pebbly to cobbly fine sands with local pockets of washed gravels.

Shallow-water sediments consisting of beach, deltaic and nearshore deposits are evident throughout much of the area from Cathcart (southwest of Brantford) to Copetown (northeast of Brantford). Beach deposits are relatively rare, with the best development in the form of a bar deposit of sand and gravel at St. George. Deltaic deposits occur mainly in the intermoraine areas west of Brantford, in the Copetown area, at Brantford, and in an area just east of Dunnville in the extreme southern part of the basin. The deltaic deposits on the eastern side of the Galt Moraine consist largely of sands washed from the Wentworth Till. Thicknesses of deltaic and nearshore deposits vary from a few feet to approximately 80 feet near the Brantford airport.

(ii) Till

Glacial till consists of deposits laid down under or at the terminus of an ice sheet. These materials are characteristically poorly sorted, compact and generally continuous over large areas.

The oldest surficial till deposit in the basin is the Catfish Creek Till located in a limited area south of Plattsville. It is a hard, compact, stoney silt till and is usually buried by younger glacial sediments throughout the rest of the basin.

The surficial tills in the northern and western portions of the basin have been identified as Tavistock, Mornington and Elma tills. They are generally sandy to silty clay tills, interbedded with glaciolacustrine silts and clays in some areas.

Port Stanley Till, which composes the surficial till in the central portions of the basin, is a stoney to bouldery, sandy silt till. This material overlies outwash and glaciolacustrine deposits throughout much of the area and in some sections is interlayered with lacustrine silts and clays.

Wentworth Till is found in the Paris and Galt moraines, which extend across the basin from northeast of Guelph to south of Brantford. This till is a stoney, sandy silt till deposited in association with the extensive kame and outwash materials associated with these two moraine complexes.

(iii) Lacustrine Silt and Clay

The only extensive deposits of fine-grained surficial lacustrine materials is located in the southern portions of the basin. These materials consist mainly of laminated to varved silts and clays, with minor sands, generally becoming more clayey toward the southeast. The sediments vary from approximately 10 feet thick in the extreme southern portions of the basin, to more than 100 feet thick south of St. George.

(iv) Bedrock Outcrops

Outcroppings of the Guelph Formation are extensive eastward from Sheffield where the overburden is thin or non-existent. Outcrops also occur along Fairchild Creek, Spencer Creek, the Speed and Elora rivers, the Grand River between Galt and Preston, and at the Elora Gorge in the Elora-Fergus area.

2.3.2 Overburden Thickness

Generally, the overburden increases in thickness from east to west (Figure 2.3). The Pleistocene deposits throughout much of the eastern half of the basin are generally 0 to 50 feet thick. Thicknesses of greater than 100 feet, with local variations up to 200 feet, occur in Erin, Eramosa and East Garafraxa townships, in the area from Cambridge to south of Guelph, and north and northeast of Brantford. This appears to be attributed to the greater deposition of materials in the vicinity of the Orangeville Moraine, the Paris and Galt moraines, and as a result of post-glacial flooding.

Maximum thicknesses of greater than 300 feet occur in Wilmot Township and in the vicinity of the towns of Milverton and Goldstone, and in the narrow bedrock valley northeast of the City of Guelph. The overburden thicknesses in Wilmot Township appear to be attributed to the increased accumulation of Late Wisconsinan deposits associated with the Waterloo Moraine, while the thick sequences of overburden in the other areas overlie bedrock valleys.

3. BASIN HYDROGEOLOGY

3.1 Introduction

The purpose of this chapter is to present general information on the water-yielding capabilities of bedrock and overburden in the basin, and consequently provide a framework for the discussion of municipal water-supply demands and how these can be met. The chapter deals with general hydrogeologic characteristics of bedrock and overlying unconsolidated materials, and the discussion focuses on areas in bedrock and overburden where well yields have been found to be generally 50 gallons per minute (gpm) or greater. Variations in natural ground-water quality in bedrock and overburden are discussed at length.

For purposes of the present investigation, only wells with the potential to yield 50 gpm or greater have been used to map areas potentially suitable for the development of municipal, industrial or large-scale agricultural water supplies. The areas with yields of 50 gpm or more have been mapped to indicate general yield conditions only, based on predominant, short-term well yields, and do not indicate site-specific yields for future development. Within any one area, ground water may be obtained from formations at different depths and of different yields, and for this reason exploratory test drilling and test pumping are an absolute necessity to evaluate the likely yields of ground water at specific sites.

Five test holes were drilled by the MOE during the course of the study in 1979 to determine the stratigraphy and lithology of high-yield deposits. The geologic and geophysical logs of these holes are shown in Appendix A.

A total of 116 water samples were taken throughout the basin to characterize waters from bedrock and overburden wells. Water samples were collected from wells completed at different depths to obtain a general profile of vertical variations in water quality. Each water sample was analysed for the following ions: calcium,

magnesium, sodium, potassium, phosphorous, sulphate, chloride, nitrate and iron. The parameters of alkalinity, hardness, total dissolved solids, specific conductance and pH were also measured.

3.2 General Hydrogeology

The quantity of water available to wells from either bedrock or overburden is dependent on the porosity and permeability of the materials. These properties are in turn related to the lithology and stratigraphy of the deposits.

The bedrock in the Grand River basin is composed mainly of limestones and dolomites. The porosity and permeability of these rocks is largely due to openings in the rock brought about by fracturing and secondary solutioning along bedding planes. The quantity of ground water available to a well is directly related to the number and size of openings. Fracturing and dissolution of the rock results in a wide variety of patterns, causing wide lateral and vertical variations in the water-yielding capacities of rocks. Hence, high and low-yield wells may exist in close proximity, both laterally and vertically.

The overburden in the basin is composed of a variety of unconsolidated materials ranging in grain size from clay to gravel, with ground water present in the pore spaces between the individual grains. The quantity of water available to wells is dependent on the shape, sorting, size distribution and packing of these grains (porosity) and the degree of interconnection of the pore spaces (permeability). Sand and gravel deposits are sufficiently porous and permeable to supply ground water at a continuous rate, and are usually the best aquifers. Fine-grained sediments, such as clays, although highly porous, are not sufficiently permeable to yield water readily to wells and for the most part act as aquitards. Poorly sorted tills, which usually have low porosities and permeabilities, also act as aquitards.

Ground-water movement within the saturated zone is governed by the distribution of fluid potential, and ground water moves from a point of high fluid potential to a point of lower potential. The water level contours in figures 3.1 and 3.2 represent lines of equal fluid potential, and ground-water flow is perpendicular to these contours. Figure 3.1 represents trends in the fluid potential (the piezometric surface) of ground water in primarily shallow bedrock, and Figure 3.2 represents trends in the elevation of the water table.

The water table and the piezometric surface in bedrock contours indicate a regional horizontal flow direction from north to south. Variations to the regional flow pattern exist due to ground-water discharges to streams, with the Grand River having the most significant effect on this flow pattern.

Vertically, ground-water movement is generally downward, except in discharge areas along the major rivers where there appears to be an upward movement of ground water from bedrock. The piezometric surface in bedrock extends above the ground level at many points along the major river valleys, resulting in flowing bedrock wells developed in these areas.

3.3 Ground-Water Yields

Yields from wells developed in bedrock and overburden formations were determined by examining the pumping tests of each well. The productivity (probable or estimated yield) of a well was determined by calculating the specific capacity of a well and multiplying this by the number of feet of water in the well. The specific capacity is the rate at which a well is pump tested at time of completion, in gallons per minute, divided by the drawdown in the well, in feet. Although the estimated yields of wells vary according to a number of factors, including the duration and rate of the pump test and the amount of penetration of the water-bearing formation by the well, the estimated values are considered valid indications of general water-yielding potentials of the formations at specific well sites.

The discussion of ground-water yields from bedrock involves basin-wide variations in estimated yields from the two major bedrock aquifers that consist of the Guelph and Amabel-Lockport formations and the Salina Formation. A similar discussion for overburden requires a lengthier description of a large number of aquifer complexes in which high-yield wells are developed.

The section on ground-water yields from overburden is divided into two parts. The first deals with four areas that are considered capable of supplying ground water for municipal use. These areas are located south of the City of Kitchener, east and south of the City of Cambridge and in the vicinity of the towns of Elmira and Fergus-Elora. The second part deals with areas that have been developed by wells with estimated yields of greater than 50 gpm, but do not appear to be readily available for development for municipal supplies because of the distance of these areas from the major municipalities. The high-yield areas in Wilmot and Woolwich townships have been developed to capacity, and the discussion of these areas is primarily for the sake of completeness of this report.

3.3.1 Yields from Bedrock

Wells with estimated yields of 50 gpm and greater exist throughout much of the northern three quarters of the basin, tapping all of the bedrock formations that subcrop within the area (Figure 3.4). The Salina, Guelph and Amabel-Lockport formations constitute the major ground-water supply sources, and because of their proximity to large water-supply demand centres, are considered the major bedrock aquifers in the basin.

(i) Guelph and Amabel-Lockport Formations

The dolomites and limestones of these formations constitute a generally high-capacity water source known as the Guelph-Amabel aquifer (Turner, 1978) throughout much of their extent in southern Ontario. Well yields are high in most areas and domestic supplies can be obtained readily throughout the aquifer. High capacity wells

are found in many areas in the aquifer as indicated by the distribution of municipal and industrial wells (Figure 3.4). Municipal water supplies for the cities of Cambridge, Guelph and various smaller towns are obtained from the aquifer. Municipal wells in Guelph have rated capacities ranging from 80 to 1300 gpm, and in Cambridge the rated capacities of the municipal bedrock wells range from 35 to 2400 gpm. Areas containing highest well yields outside of the major development areas appear to be in the vicinity of the towns of Fergus-Elora, Arthur and Dundalk, and in Puslinch, Erin, Amaranth and East Luther townships.

Depths of wells in the formation are variable, depending on the overburden thickness and penetration into the bedrock. Generally, most of the high-capacity wells are more than 100 feet deep, with wells greater than 200 and 300 feet deep situated in the vicinity of Cambridge and Guelph, and throughout the area north of the City of Guelph. Wells are generally less than 100 feet deep in Beverly Township and in the northern part of the basin. Penetration of wells into the bedrock is highly variable due to the variability of the hydrogeologic characteristics of the bedrock. Generally, the majority of domestic wells receive water from the upper 50 feet of bedrock, while municipal and some industrial wells penetrate the bedrock to depths of 100 to 200 feet. A few receive water at greater depths. For example, a municipal supply well in the Town of Arthur has been developed to a depth of 617 feet, penetrating 415 feet into the rock.

The unconsolidated materials overlying the Guelph-Amabel aquifer are, for the most part, less than 50 feet thick, with bedrock exposed in many areas in the basin (Figure 2.3). Extensive outwash deposits directly overlie the aquifer in the vicinity of Guelph, resulting in relatively high infiltration rates to the aquifer.

(ii) Salina Formation

The Salina dolomites and limestones constitute generally high-capacity water-supply sources north of Kitchener-Waterloo. Areas where well yields are estimated to be greater than 200 gpm extend from Kitchener-Waterloo northwest to the drainage basin boundary and from Moorefield eastward to the Town of Arthur.

Estimated well yields are generally less than 50 gpm throughout the southern portions of the basin, except for some limited areas along the Grand River and in the vicinity of Dunnville (Figure 3.4). The municipal wells for Caledonia obtain water from the Salina Formation, where they yield water at rates of greater than 200 gpm.

Substantial fracturing of the bedrock was encountered in test hole 4969 (South of Kitchener - Appendix A) after 3 feet of penetration. Mud circulation could not be maintained and the hole could not be advanced. Circulation was also lost after 5 feet of penetration of the bedrock at test hole 4970 (South of Kitchener - Appendix A). The fracturing at both test sites indicates a high permeability of the bedrock and consequently a good aquifer.

Depths of wells developed in the Salina Formation vary considerably due to large variations in overburden thickness. Once through the overburden, wells penetrate generally less than 50 feet into bedrock.

The Salina Formation is largely confined by aquitard materials ranging from less than 50 feet thick in the southern portions of the basin to more than 300 feet in some parts in the northern areas.

3.3.2 Yields from Overburden

The overburden consists of a thick complex of Pleistocene deposits of glacial and interglacial origin. The high-yield areas (Figure 3.5) represent yields from wells developed in permeable deposits that occur at varying depths within these materials. Four of these areas appear to have a potential to supply ground water for

municipal use: south of the Kitchener, east and south of the City of Cambridge and in the vicinity of the towns of Elmira and Fergus-Elora.

The high-yield areas of interest that are located south of Kitchener extend from southern Waterloo Township, through much of North Dumfries Township west of the Grand River, to Ayr. The complex stratigraphy of the overburden within these areas results in a high degree of lateral and vertical variability in the lithologies of the high-yield deposits. Generally, they occur at depths ranging from 45 feet to 170 feet, with an average depth of about 110 feet. They consist of fine to coarse sands and gravels that generally vary from 10 to 30 feet thick, grading laterally and vertically into sequences of clays, silts and fine sands interlayered with tills. Test hole 4969 (Appendix A), located one concession north of Roseville, revealed 36 feet of fine to medium sands and gravels with interlayered silts and some clays at depths from 60 feet to 96 feet. This sequence is confined by 60 feet of lacustrine sediments. At test hole 4970 (Appendix A), located one concession to the east, the permeable sequence is about 45 feet thick and extends from a depth of 76 feet to about 120. It consists of fine sands that grade into a sequence of interlayered fine to coarse sands and gravels, and is confined by a stoney silt till interlayered with lacustrine sediments. The complex slopes generally in a southerly to southeasterly direction from Waterloo Township to Ayr. In the vicinity of Ayr, the complex occurs as medium to coarse sands and gravels that generally extend from 58 feet deep to about 102 feet deep.

Estimated yields from wells developed in the complex are generally 50 to 100 gpm in northwestern North Dumfries Township and southern Woolwich Township. These well yields are based on short-term pumping tests, at low pumping rates, and consequently the estimated yields may not be indicative of the true potential of the deposits. High-yield wells that tap the complex in the central portions of North Dumfries Township have estimated yields of generally between

100 and 200 gpm. Estimated yields from wells developed in these deposits in the vicinity of Ayr are generally in the order of 50 to 100 gpm. An exception is industrial well 22/63 (Figure 3.5), which has an estimated yield of greater than 700 gpm.

A number of wells in the vicinity of Ayr tap a sequence of fine to coarse gravels that occur at depths of about 120 feet to 174 feet. The deposits appear to be thickest in the vicinity of Ayr, up to 100 feet in some places, and appear to thin or grade into finer materials away from the town. Estimated yields of the wells developed in this sequence are generally greater than 200 gpm. The two municipal wells for Ayr tap these deposits and have rated capacities of greater than 300 gpm each.

Wells 85/249 and 59/210 (Figure 3.5) located just south of Highway 401, and well 58/222 located south of Ayr, tap sand and gravel deposits at depths of greater than 200 feet. These materials may be comparable to the lower sequence in Ayr. Estimated yields from these wells are greater than 200 gpm.

The high-yield area located in Puslinch Township, east of the City of Cambridge, represents yields from wells tapping a fairly extensive sequence of outwash materials. The deposits form an aquifer that consists of about 25 to 30 feet of medium to coarse sands and gravels. They appear to directly overlie and are believed to be connected with bedrock in many places. The deposits are generally 80 to 120 feet deep and are confined by lacustrine sediments and till(s). They appear to pinch out to the north and south and become thin and discontinuous to the northeast. Estimated yields of high-capacity wells developed in the aquifer are generally greater than 200 gpm.

The deposits in Puslinch Township appear to extend to the southwest, through North Dumfries Township east of the Grand River, and along the Grand River to northeast of the Town of Paris.

The deposits appear to thin toward the south, generally ranging from a few feet to approximately 20 feet thick. They are confined by a complex of lacustrine deposits interlayered with till(s). Estimated yields from high-capacity wells vary from 50 gpm to between 100 and 200 gpm. High yield wells in the vicinity of Shades Mills, three of which are municipal wells for the City of Cambridge, tap a sequence of sands and gravels that are up to 50 feet thick. The materials are overlain by a thick sequence of kame and outwash materials. Like the deposits composing the aquifer in Puslinch Township, the aquifer in the vicinity of Shades Mills directly overlies and is believed to be connected to bedrock. Estimated yields for the high-capacity wells developed in these deposits are greater than 200 gpm.

The high-yield area located along the Grand River northeast of Paris consists of a sequence of basal outwash sands and gravels that may be an extension of the basal materials to the north. They occur at depths of 61 to 88 feet, are generally less than 20 feet thick, and pinch out west of the Grand River. They appear to extend toward the east to about well 22/77 (Figure 3.5). Two municipal wells for Paris tap these deposits and have estimated yields of up to about 800 gpm.

Wells in the vicinity of the town of Elmira tap permeable deposits contained in a complex of interglacial sediments, possibly kame materials, interlayered with till(s). Most of the high-yield wells are developed to a depth of 100 feet ± 30 feet in medium to coarse sand and gravel deposits that appear to directly overlie the bedrock in most places. The deposits dip to the south, following the topographic trends of the underlying bedrock. Sequences of sands and gravels up to 80 feet thick occur in the immediate vicinity of Elmira, grading into clayey gravels north of the Conestogo River and pinching out to the east, west and north. Estimated yields from wells are generally greater than 200 gpm. The municipal wells for Elmira tap these deposits. Wells developed in the thinner or finer deposits to the east and west of the town have estimated yields of between 50 and 100 gpm.

Wells in the vicinity of Floradale north of Elmira tap sand and gravel deposits at variable depths from 40 to 143 feet. The materials appear to exist as lenticular deposits contained in about 90 feet of lacustrine and glacial sediments. Estimated yields from wells developed in these deposits range from 100 to 200 gpm. Two wells in the northern part of the area tap a deeper deposit of fine to coarse gravels that is about 30 feet thick. These materials may be resting directly on the bedrock and are confined by about 140 feet of till and lacustrine deposits. Estimated yields from these wells are greater than 200 gpm.

The narrow, high-yield area through Fergus and south of Elora is due to permeable deposits that occur in a northeast to southwest-trending bedrock valley. Karrow (1979) mapped the valley as extending from east of Kitchener to north of the Elora-Fergus area. The valley may extend as far north as Grand Valley. Hilton (1978) conducted an extensive geophysical survey of the valley from south of Lake Belwood to Inverhaugh and drilled four test holes east of Elora to assist in the interpretation of the geophysical information. Two of the holes, UW1 and UW2 (Figure 4.7) were located over the axis of the buried valley. Test hole UW1 revealed a deposit of stones and gravel at a depth of 172 feet. The drilling continued to a depth of 178 feet, at which point the operation was discontinued due to the loss of mud circulation. Test hole UW2 revealed a deposit of large stones and gravel at a depth of 218 feet. Circulation was again lost and drilling was discontinued at 222 feet. The permeable materials in both test holes are overlain by a complex of tills and a thick sequence of consistently soft clayey sands. Test hole 7047 (Appendix A) revealed a sequence of fine to coarse sand, grading into fine to coarse gravels, starting at a depth of 181 feet. Permeability of the material is quite high as there was substantial mud loss while penetrating the formation. The aquifer at this location is confined by a thick sequence of interlayered silts, clays and fine sands, underlying about 68 feet of till. A deposit of clay and silt, 3 feet thick, separates the permeable strata from the bedrock.

Following is a discussion of high-yield areas that represent permeable deposits from which ground-water development is less likely in the near future due to their distance from major municipalities.

The high-yield areas in Peel, southern Woolwich, Wellesley, northern Wilmot and Mornington townships consist mostly of deep basal outwash deposits that either directly overlie bedrock or are separated from bedrock by a thin clay layer. Thicknesses of up to 40 feet have been reported in some wells, but the deposits appear to average 10 to 15 feet, with thicknesses generally increasing where the materials directly overlie bedrock depressions. The aquifers in these areas occur generally as permeable strata, interlayered with silts and clays, and are generally confined by over 200 feet of glacial and interglacial sediments. Estimated yields from high-capacity wells developed in these deposits range from about 50 to 100 gpm, to greater than 200 gpm. Several wells tap sand and gravel deposits at shallower depths within many of the high yield areas.

Deposits consisting of fine to medium sand occur in the vicinity of St. Clements. They are generally 100 to 130 feet deep and are confined by tills and interglacial sediments. Test hole 4968 (Appendix A) revealed a sequence of fine sands and silts with seams of medium to coarse sand extending from a depth of about 115 feet to 148 feet. These deposits are overlain by interlayered silts, clays and sands. The basal sands and gravels expected in the area were not encountered in the test hole. From on-site inspection of samples and the low mud loss while penetrating the upper formation, it appears that the materials are suitable only for low-yield domestic use.

Sand and gravel deposits interlayered with till occur at varying depths above basal deposits in the high-yield areas between the towns of Milverton and Wellesley. Estimated well yields from these deposits generally range from 50 gpm to greater than 200 gpm.

Most of the high-yield wells in Wilmot and Waterloo townships obtain water from deposits contained in a complex sequence of interglacial materials that occur at varying depths. The major water-producing sequences are generally thickest immediately west of Kitchener-Waterloo and in the vicinity of Baden. The deposits vary in depth and thickness throughout central Wilmot Township and appear to extend throughout much of Waterloo Township. The water-bearing deposits in Waterloo Township consist of a complex of permeable sequences ranging from 15 to 70 feet thick. These deposits appear to be overlain by tills, kame and outwash deposits, and overlie till or rest directly on bedrock. They are developed extensively by domestic, industrial and municipal wells. The municipal wells for the City of Kitchener-Waterloo and the town of Baden tap permeable sequences in this complex.

Wells 35/312, 31/222 and 50/240, located northeast of Everton in Erin Township (Figure 3.5), have been developed in permeable gravels in the bedrock valley that extends northeast from the City of Guelph. The aquifer is generally overlain by ablation deposits associated with the Paris and Galt moraines and older tills. The gravels may extend along the full length of the valley, but this is difficult to determine at the present time due to the lack of data. Estimated yields of wells developed in the deposits are greater than 200 gpm.

High-yield wells located in South Dumfries Township, in the vicinity of Pinehurst Lake, tap permeable deposits consisting of medium sands and gravels at depths of 127 to 222 feet. Wells 40/128 and 54/127 (Figure 3.5) tap a deposit of medium sand and gravel at a depth of 127 feet. Estimated yields for the two wells are 68 gpm at well 54/127 and 145 gpm at well 40/128. Well 70/170 taps a gravel deposit that extends from 170 to 185 feet and appears to be confined by a stoney sand till. The estimated yield of this well is 158 gpm. Well 57/222 taps a deep basal deposit at 220 feet that consists of gravel, which may directly overlie the bedrock. The estimated yield of the well is 892 gpm.

High-yield wells located in South Dumfries Township, between the Grand River and St. George, tap medium to coarse sands and gravels that vary in depth from 52 feet to 172 feet. These deposits occur as permeable strata interlayered with clays, silts, fine sands and tills. The deposits are generally confined by a stoney, sandy silt till that extends from ground surface to about 80 feet. Thicknesses of the deposits vary from less than 10 feet to about 20 feet. Test hole 2755 (Appendix A) revealed 6 feet of fine, compact sand at a depth of about 92 feet, confined by a thick stoney, sandy silt till. Estimated well yields in these areas are generally more than 200 gpm.

High-yield wells north of St. George tap gravel deposits at depths varying from 64 feet to 210 feet. Wells 64/210 and 55/178 (Figure 3.5) appear to tap a deep gravel deposit that directly overlies bedrock. The deposit appears to be between 10 feet and 20 feet thick and is generally overlain by a complex of sediments consisting of fine sands, silts, clays and tills. The other high-yield wells north of St. George tap gravel deposits within shallower interglacial sequences consisting of medium to coarse sands and gravels. Estimated yields from wells developed in these two sequences range from 50 to 200 gpm.

High-yield wells south of St. George tap medium sand and gravel deposits ranging from beteen 57 and 133 feet deep. The two high-yield wells in town are developed in coarse sand and gravel interlayered with silt. The aquifer is about 10 feet thick and is overlain by clay. Estimated well yields range from 100 gpm to 200 gpm.

There are several high-yield overburden formations of relatively limited lateral and vertical extent located in the southern portions of the basin. High-yield wells located west and northwest of Paris, west of Brantford, in the area southeast of Victoria Mills, and near Cayuga and north of Dunnville, are developed in sand and gravel deposits that are contained in sequences of interglacial sediments.

These deposits are generally 15 to 20 feet thick and range from 16 feet to over 164 feet deep. They appear to directly overlie the bedrock in many places and have estimated well yields from 50 gpm to greater than 200 gpm.

The high-yield areas southwest of Brantford consist of thick sequences of outwash gravel, beach and nearshore lacustrine sediments. These deposits vary from thick unconfined sequences of fine to medium sand, with wells developed generally at depths of less than 10 to approximately 30 feet, to sand and gravel deposits 30 to 180 feet deep. Estimated well yields range from 50 gpm to 200 gpm.

The high-yield area in the vicinity of Burford consists of sand and gravel deposits that grade into sand and silt at depth. Depths of wells developed in the materials vary from 21 feet to 75 feet and estimated well yields vary from 50 gpm to 200 gpm.

3.4 Ground-Water Quality

The chemical composition of ground water is subject to changes relative to its movement through varying geologic environments. These changes are functions of a number of interacting factors: the initial chemical composition and temperature of the infiltrating water, the minerology of the medium through which ground water is moving, and the contact time between the medium and the water.

When ground-water chemistry is viewed on a regional scale, certain broad relationships between concentrations of major ions in ground water and the geologic environment can be predicted. For example, ground water in calcite-rich deposits such as in limestone or dolomite generally has high calcium and bicarbonate concentrations.

There is also a relationship between hydrochemistry and ground-water movement. Ground-water quality generally deteriorates with distance travelled from the ground surface. Water samples collected in the same area, but at different depths, may therefore have different chemical characteristics.

Ground water to be used for municipal uses should meet certain inorganic chemical criteria to be safe for consumption and to minimize or avoid expensive water treatment. The most common inorganic parameters of concern, and their permissible criteria, are:

Sulphate	-	250 mg/L
Chloride	-	250 mg/L
Iron	-	.3 mg/L
Nitrate (asN)	-	10. mg/L
Total Dissolved Solids	-	500 mg/L

In addition, the classification of hardness of water is:

60 mg/L ($CaCO_3$)	-	soft
61-121	-	moderately soft
121-180	-	hard
180	-	very hard

There are generally no criteria for hardness, as this depends on individual preferences, but some degree of water softening may be desirable for very hard water.

Concentrations in excess of the permissible criteria may affect the use of the water for domestic supplies. A brief discussion of the parameters and the affect that an excess concentration of each may have on domestic uses follows.

The criterion for sulphate is based primarily on its laxative effects on unacclimatized users. Consumption of water with sulphate concentrations in excess of permissible limits does not, therefore, pose a serious health hazard.

The criterion for chloride is based on the palatability of the water. Concentrations at which water will taste salty vary from person to person, but this limit is usually taken to be 250mg/L.

Iron is not considered toxic at concentrations usually found in ground water, but it is considered objectionable in domestic water supplies because of the bitter taste it can give to water and because it can stain porcelain plumbing fixtures, laundry, etc.

Nitrate concentrations in excess of the permissible level is generally considered dangerous for use in infant formulae due to the possibility of the development of methemoglobinemia, which can be fatal if not treated properly. Generally, adults can tolerate much higher concentrations.

Total dissolved solids (TDS) represent the sum of dissolved cations and anions in water. The permissible criteria for TDS is somewhat arbitrary and water with much higher concentrations can be consumed without harm. High TDS in itself does not restrict the use of the water for domestic supply. The dominant ions in solution must be identified to determine the potential effects on water users.

Water hardness is due to the concentration of magnesium and calcium ions in solution, which affect the cleaning power of soap. The reduction of water hardness is dependent primarily on economic and convenience considerations.

3.4.1 Quality in Bedrock

Sample sites were randomly selected from the high-yield bedrock wells on Figure 3.4. Basin-wide trends in the quality of water from these wells can therefore be assumed to be generally indicative of ground-water quality from different formations in the basin (Figure 3.6).

The Salina Formation is composed mainly of limestones, dolomites, shales and evaporite deposits. The evaporites occur in beds of varying vertical and lateral extent throughout the entire thickness of the formation and consist mainly of anhydrite ($CaSO_4$), gypsum ($CaSO_4 \cdot 2H_2O$) and salts such as halite (NaCl) and sylvite (KCl). When ground water moves through these deposits, dissolution of anhydrite, gypsum and the various salts occurs, resulting in high ionic concentrations of calcium, sulphate, sodium, potassium and chloride, and consequently in high levels of total dissolved solids.

The Guelph and Amabel-Lockport formations are composed mainly of limestones and dolomites. Although evaporite deposits exist within these formations, they are not significant. The major minerals composing these rocks are calcite ($CaCO_3$) and dolomite ($CaMg(CO_3)_2$). Generally, as ground water moves through these formations, dissolution of calcite and dolomite occurs, resulting in high concentrations of bicarbonate and calcium ions. The solubilities of calcite and dolomite are generally lower than those of anhydrite and gypsum, and this results in lower levels of total dissolved solids than in ground water from the Salina Formation.

When the ionic concentrations of ground water from bedrock are examined, a general trend emerges. Throughout most of the northern and eastern portions of the basin, ground water is of calcium-bicarbonate type (calcium and bicarbonate are dominant ions), while in the western and southern parts of the basin calcium-sulphate waters are most common (dominant ions are calcium and sulphate). In addition, total dissolved solids content in waters sampled from bedrock in the western and southern parts of the basin is generally higher. These general trends reflect the minerology of the bedrock formations through which the ground water is moving.

(i) Suitability for Domestic Use

(a) Sulphate: Of the 49 water samples taken from bedrock wells, 18 have sulphate concentrations well in excess of the permissible level of 250 mg/L (Table 3.1, Figure 3.6), with a mean value of 1282 mg/L and a maximum concentration of 2209 mg/L from well 258. This well is located northeast of Dunnville. All of the wells with high sulphate concentrations are developed in the Salina Formation.

(b) Chloride: One well, number 1329 located in Brant County east of the City of Brantford, contains water with chloride concentrations in excess of 250 mg/L. High concentrations of other ions in this well, notably sodium, calcium and sulphate, plus high TDS, possibly indicates ground-water movement through evaporite deposits.

(c) Iron: Nearly one-half of the samples taken have iron concentrations in excess of the recommended limit of 0.3 mg/L. Concentrations in water from these wells range from .4 mg/L to 33 mg/L, with the highest concentrations occurring in water from wells developed in the Salina Formation.

(d) Nitrate: Waters from wells 516, 2186, 2731 and 4342 (Figure 3.6) have nitrate concentration greater than the recommended limit of 10 mg/L. The concentrations of nitrate from these wells range from 12 mg/L at well 2186 to 20 mg/L at well 516 (Table 3.1, Figure 3.6). The site-specific reasons of these high concentrations is not known.

(e) Total Dissolved Solids: Twenty-two of the 49 bedrock wells sampled contain water with more than 500 mg/L TDS. In most of these cases, the ground water contains high sulphate and calcium concentrations. Calcium does not limit the use of water for domestic supply, but sulphate may have a laxative affect on unacclimatized users. All of these wells are developed in the Salina Formation.

(f) Hardness: All of the water samples collected from bedrock can be classified as "very hard". The mean for the 49 samples is 681 mg/L, with a maximum concentrations of 2520 mg/L found in well 338 (Figure 3.6). In all cases calcium is the main ion contributing to the high hardness.

3.4.2 Quality in Overburden

Variations in dissolved minerals in ground water from overburden are illustrated on Figure 3.7. Sample sites were randomly selected from wells developed in overburden deposits at varying depths throughout the basin in an attempt to define the general lateral and vertical trends in water quality. Generally, waters from deep overburden deposits that are believed to be connected with the underlying Salina Formation have high sulphate concentrations and therefore may pose problems for domestic use.

Of the 67 wells sampled, 49 contained water of calcium-bicarbonate type, while the rest were of calcium-sulphate type. All but three of the water samples with high concentrations of sulphate are from

basal aquifers that directly overlie and are thought to be connected with bedrock. The high sulphates may therefore be attributed to ground water moving through the bedrock and into the overlying unconsolidated materials. The sources of high sulphates in the three samples obtained from shallow overburden deposits are not known.

(i) Suitability for Domestic Use

(a) Sulphate: Fourteen of the 67 water samples taken have sulphate concentrations in excess of the permissible level, with a mean value of 980 mg/L and a maximum concentration of 1840 mg/L from the water in well 244 (Table 3.2, Figure 3.7). Most of these wells are developed in deep overburden deposits that appear to be connected with the Salina Formation. Wells 905, 1478 and 3382 (Figure 3.7) are developed in shallow deposits that occur within 50 feet of the ground surface. The reasons for the high sulphate concentrations in these three wells are not known.

(b) Chloride: One well, number 870, located west of the Town of Dunnville, contains water with a chloride concentration of 481 mg/L. High concentrations of other ions in this well water, notably sodium, calcium and sulphate, plus high total dissolved solids, possibly indicates ground-water movement from the Salina Formation into the overlying unconsolidated materials.

(c) Iron: Ground water from 33 of the 67 wells sampled have iron concentrations in excess of the recommended limit. Concentrations in these wells range from .4 mg/L to 9.1 mg/L and are found in overburden deposits at various depths (Figure 3.7).

(d) Nitrate: Ground waters from wells 752, 1087, 1955, 3181, 3408 and 837 have nitrate concentrations greater than the recommended limit. Wells 1087 and 3408 are developed in shallow deposits that occur within 50 feet of ground surface. Wells 752, 1955, 3181 and 837 are developed in sediments ranging from 60 feet to 243 feet deep (Figure

3.7). Concentrations in these wells range from 11 mg/L to 36 mg/L and are probably the result of contamination from local sources.

(e) Total Dissolved Solids: Twenty-six of the 67 overburden wells sampled contain water with more than 500 mg/L TDS. Levels in these wells range from 535 mg/L to 3540 mg/L. In most cases, ground water from these wells contains high sulphate and calcium.

(f) Hardness: All but one of the water samples collected can be classified as "very hard". Well 2312, located in Puslinch Township, contains water that can be classified as "hard". The mean for the 67 samples is 500 mg/L, with a maximum concentration of 2120 mg/L found in well 870 (Figure 3.7). In all cases, calcium is the main ion contributing to the high hardness values.

4. MUNICIPAL GROUND-WATER SUPPLIES

4.1 Introduction

Within the Grand River basin, ground water is the most important source of municipal water supplies. In 1979, there were 19 communities utilizing ground water for municipal supplies, and 4 communities for which ground-water surveys have been undertaken in anticipation of ground-water supply systems in the future. By contrast, there are only 3 communities with municipal supplies based on surface-water sources. These communities are Brantford, Cayuga and Dunnville.

In 1977, the total average daily water consumption for the 19 communities using ground water was 41.302 million gallons per day (mgd). The total average daily demands, based on a medium population growth, are projected to be 70.408 mgd in 2001 and 107.993 mgd in 2031. These projected totals include the 4 communities that have no municipal water-supply system at the present time, but are expected to be utilizing ground water in the future. Thus it is apparent that ground water is now, and will continue to be for some time, an important source of municipal water supplies in the Grand River basin.

The purpose of this chapter is to evaluate the potential of ground water for future development by each community within the Grand River basin that has been, or may be expecting to utilize ground water in the future. More specifically, the present and future (to the year 2031) water requirements of each community have been determined and compared to known supplies and, where the proven supplies have been deemed to be inadequte to meet future demands, potential sources of additional ground water have been identified.

The supply/demand situation of Kitchener-Waterloo warranted special attention because the twin cities are the largest municipal users of ground water in the basin, and because they have experienced water shortages during summer peak-demand periods in the past. The analysis of potential ground-water supplies for Kitchener- Waterloo includes an evaluation of specific water supply options in terms of design criteria, project costing, and environmental and social impacts of each project.

The 1977 municipal water demand of each community in the basin was determined from data on file with the Water Resources Branch of the Ministry of the Environment (MOE) and from direct correspondence with each community. The 1979 capacity of each of the systems was also estimated from these sources, as well as from other available (consultants) reports commissioned by the MOE or by the municipalities concerned.

Future water demands were based on 1977 per capita requirements as reported by each community, applied to projected population changes. It is recognized that the per capita requirements can vary (Fortin and Veale, 1981), especially when water conservation measures are applied, and consequently the 1977 demand estimates represent one set of approximations for purposes of this report only. The population projections were generated by the Grand River Conservation Authority (Veale, 1981) and were based on past trends in births, deaths, and migration statistics. Three such projections (low, medium, and high growths) were derived for each community. Appendix B lists these projections for the cities, towns and villages in the basin, and Appendix C lists the projections for the unincorporated communities which are presently using or may be expecting to use ground water in the future.

For those communities with insufficient supplies to meet the expected future demands, areas with potential for future ground-water developments were identified. These areas were determined from information contained on ground-water yield maps presented in this report, augmented by detailed hydrogeologic information derived from water well records on file with the MOE.

The 23 communities for which ground-water evaluations were made are grouped and discussed in four categories:

1. communities with sufficient supplies to meet the projected demands by the year 2031;

2. communities with insufficient supplies to meet the 2031 demands;

3. communities with no existing municipal water systems; and,

4. water supply/demand situation of Kitchener-Waterloo, including an appraisal of proposed water-supply options for the twin cities.

It is important to note that the projected demands for 2001 and 2031, which were compared to existing supplies to determine if the supplies were sufficient or not, refer to average daily demands based on medium population projections. It was assumed that maximum daily demands will be satisfied from storage and/or aquifer over-pumping for short periods of time. However, this may not be practical and the design of new pumping facilities must allow for maximum-day demands.

A summary of present (1979) municipal supplies and projected future demands for each community discussed in this report is presented in Table 4.1. Since Brantford, Cayuga and Dunnville obtain their municipal supplies from surface water sources, they are not included in this summary or the following text.

Table 4.1 Summary of Present Municipal Supplies and Future Projected Demands by the years 2001 and 2031

Community	1977 Average Daily Consumption (mgd)	1979 System Capacity (mgd)	2001 Medium Population Projection	2001 Average Daily Demand (mgd)	Additional Supplies Needed by 2001** (mgd)	2031 Medium Population Projection	2031 Average Daily Demand (mgd)	Additional Supplies Needed by 2031** (mgd)	Potential For Future Development
ARTHUR	0.200	0.633	2,200	0.270	NONE	3,111	0.382	NONE	GOOD
AYR	0.110	1.037	2,283	0.189	NONE	3,277	0.272	NONE	GOOD
BADEN-NEW HAMBURG	0.409	1.684	5,941	0.546	NONE	8,526	0.783	NONE	GOOD
CALEDONIA	0.323	2.560	9,750	0.858	NONE	15,824	1.392	NONE	GOOD
CAMBRIDGE	8.577	16.990	124,474	14.940	NONE	178,662	21.440	4.450	GOOD
DUNDALK	0.113	0.432	1,854	0.185	NONE	3,350	0.335	NONE	GOOD
ELMIRA-ST. JACOBS	1.680	4.610	10,389	2.214	NONE	14,911	3.178	NONE	GOOD
ELORA	0.247	0.668	5,355	0.536	NONE	10,844	1.084	0.416	GOOD
FERGUS	0.615	1.764	11,133	1.147	NONE	22,810	2.351	0.587	FAIR
GUELPH	9.500	25.810*	115,456	15.580	NONE	209,133	28.230	2.420	FAIR
KITCHENER-WATERLOO	18.177	32.648	316,417	31.641	NONE	451,577	45.157	12.509	GOOD
MARYHILL	0.008	0.072	548	0.031	NONE	786	0.045	NONE	GOOD
MILVERTON	0.129	0.315	2,057	0.189	NONE	3,257	0.299	NONE	GOOD

ble 4.1 (continued)

mmunity	1977 Average Daily Consumption (mgd)	1979 System Capacity (mgd)	2001 Medium Population Projection	2001 Average Daily Demand (mgd)	Additional Supplies Needed by 2001** (mgd)	2031 Medium Population Projection	2031 Average Daily Demand (mgd)	Additional Supplies Needed by 2031** (mgd)	Potential For Future Development
RIS	1.064	2.880	8,800	1.126	NONE	10,920	1.398	NONE	GOOD
ATTSVILLE	N/A	0.374	639	0.064	NONE	861	0.086	NONE	GOOD
CKWOOD	N/A	0.864	1,817	0.102	NONE	3,290	0.184	NONE	GOOD
. GEORGE	0.150	1.800	1,766	0.284	NONE	3,803	0.612	NONE	GOOD
RFORD	NO SYSTEM		1,191	0.119	N/A	1,383	0.138	N/A	GOOD
AYTON	NO STSTEM		1,179	0.118	N/A	1,843	0.184	N/A	GOOD
AND VALLEY	NO SYSTEM		1,739	0.174	N/A	3,150	0.315	N/A	GOOD
LEM	NO SYSTEM		953	0.095	N/A	1,284	0.128	N/A	GOOD

This does not include artificial recharge during the summer months

Additional supplies refer to annual average-day demands, whereas the design of a water-supply system must be based on adequate pumping capacity to meet maximum-day demands. For design puposes, the maximum-day demands are often estimated to be 1.6 times the average-day demands.

4.2 Communities with Sufficient Supplies Until 2031

The communities with sufficient supplies to meet the average daily demands, based on a medium population projection in the year 2031, are:

Arthur	Dundalk	Paris
Ayr	Elmira-St. Jacobs	Plattsville
Baden-New Hamburg	Maryhill	Rockwood
Caledonia	Milverton	St. George

An evaluation of the present and future water supply/demand situation for each community follows.

4.2.1 Arthur

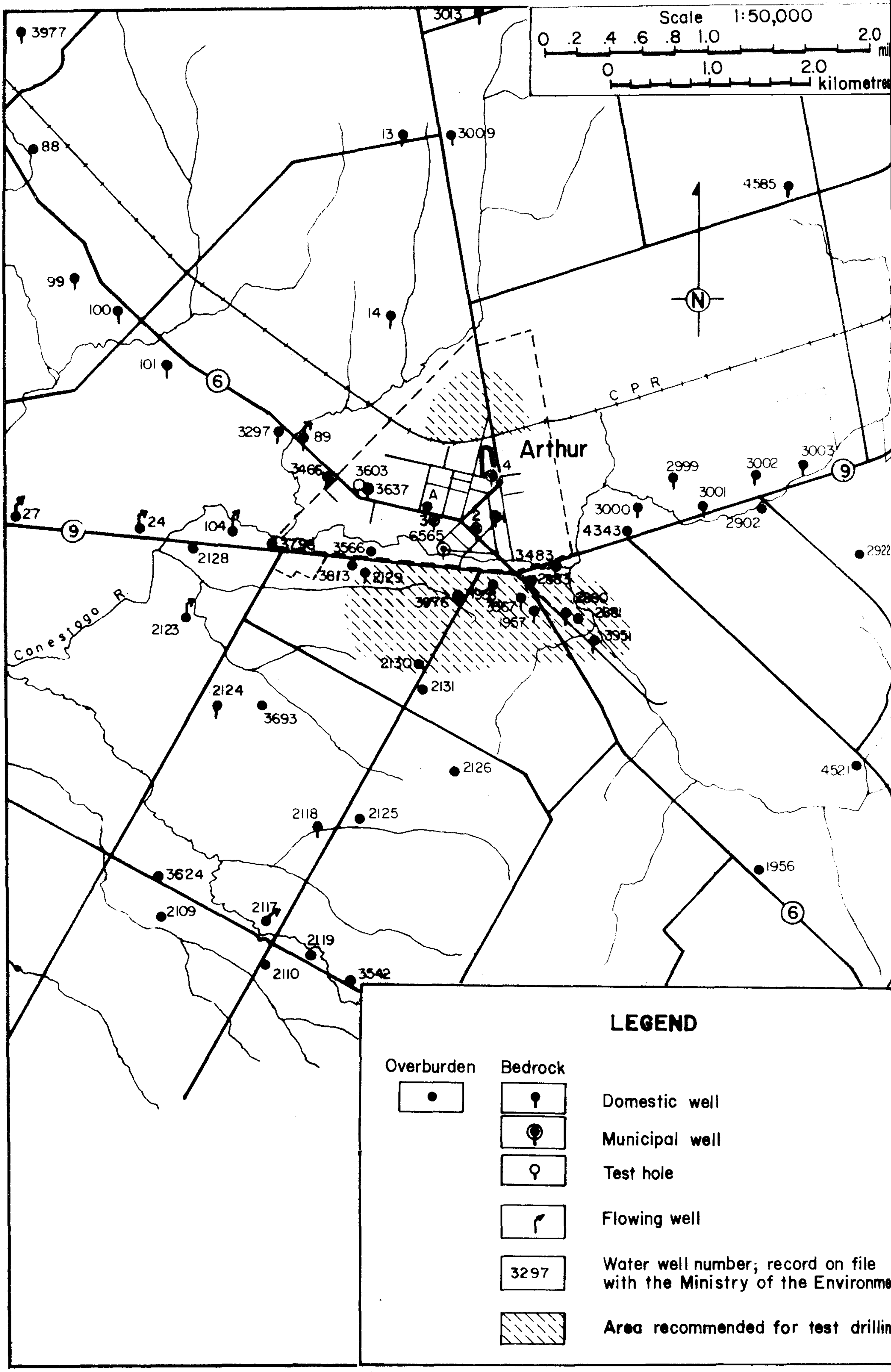
(i) Location and Physical Description

The Village of Arthur is located in the northcentral portion of the Grand River basin, mid-way between Drayton and Grand Valley, approximately 30 miles north of Kitchener.

Overburden thickness in the area varies from 200 - 250 feet, and consists primarily of clayey, stoney silt till at the surface, interbedded at depth with layers of clay, sand and gravel of various thicknesses. The underlying bedrock consists of approximately 10 - 50 feet of shale of the Salina Formation, overlying dolomite and limestone of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system of Arthur consisted of five bedrock wells, with a combined rated capacity of 0.446 mgd, and one standby bedrock well rated at 0.187 mgd (Figure 4.1). The natural water quality of these wells is suitable for municipal use and no treatment is in effect at the present time. Some elevated sulphate and iron concentrations have been reported in the area but these are generally within acceptable criteria for potable supplies.



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The 1977 demand on this system was 0.200 and 0.390 mgd for average and maximum daily consumptions, respectively. Approximately 30 percent of these daily consumptions was attributed to industrial use by Bell Thread Limited. This industry is expected to increase its water consumption to about 40 percent of the total daily demand following a proposed plant expansion.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	2,038	2,200	2,362
average daily demands (mgd)	0.250	0.270	0.290
maximum daily demands (mgd)	0.488	0.527	0.566
2031 population projections	2,530	3,111	3,692
average daily demands (mgd)	0.311	0.382	0.454
maximum daily demands (mgd)	0.606	0.745	0.884

Note: The population projections for other years for Arthur are shown in Appendix B.

(iv) Potential Future Supplies

The existing supply (including the standby well) of 0.633 mgd is more than sufficient to satisfy the projected average daily demand by the year 2031. If required, it is likely that additional water supplies could be developed almost anywhere in the vicinity of Arthur. The "Ground-Water Yields From Bedrock" map (Figure 3.4) indicates potential well yields from bedrock in this area in excess of 200 gpm (0.288 mgd). An MOE ground-water survey of Arthur (Steltner, 1976) determined that although the local sand and gravel lenses can yield sufficient quantities of water for individual domestic uses, they generally exhibit low potential for municipal development. Further, the same report stated that yields of up to 400 gpm (0.576 mgd) are likely from the Guelph Formation bedrock, and that the chemical quality of water from the top 200 feet of this formation should be suitable for municipal use.

Two areas are recommended for future test drilling (Figure 4.1). These areas are presently accessible for test drilling and well development, and are probably sufficiently removed from the existing municipal wells to minimize well interference.

The northern test-drilling area, northeast of Highway No. 6, has approximately 200 feet of overburden and a main water-bearing horizon within the Guelph Formation is found at a depth of approximately 300 feet. The water quality in the bedrock is expected to be good, with some elevated iron concentrations possible.

The second recommended area, south of Highway No. 9, has about 250 feet of overburden, with water-bearing deposits at 50 and 150 feet in the overburden. Water-yielding bedrock is encountered at about 300 feet. Both overburden and bedrock should be tested for potential municipal development. The possibility of elevated sulphate concentrations in bedrock increases with distance west of Highway No. 6.

(v) Summary

The 1979 municipal water supply of Arthur consisted of five bedrock wells, with a combined rated capacity of 0.446 mgd, and one standby bedrock well rated at 0.187 mgd. This supply appears to be sufficient to meet the average daily demand based on a medium population projection in 2031. The Guelph Formation bedrock in this area has good potential for high-yield development and there should be little difficulty in securing additional municipal water supplies. Two areas suitable for test drilling for municipal supplies have been identified.

4.2.2 Ayr

(i) Location and Physical Setting

Ayr is located at the confluence of the Nith River and Cedar Creek, approximately 12 miles south of Kitchener.

General overburden thickness in the area varies from 200 - 250 feet and consists mainly of thick beds of silt, sand, and gravel, which are interbedded with clay and glacial till. The surficial geology in the area consists mainly of kame and outwash gravels and lacustrine sands. Both the surficial sands and gravels, and those found at depth, have been reported to attain thicknesses of up to 100 feet in the vicinity of Ayr. The underlying bedrock is Silurian dolomite of the Salina Formation.

(ii) Existing Supply

The 1979 municipal water-supply system of Ayr consisted of two overburden wells with a combined rated capacity of 1.037 mgd (Figure 4.2). Overburden ground water in this area is of good quality, with the exception of some elevated iron concentrations that appear to increase with depth. The 1977 average daily water demand was 0.110 mgd. Information concerning maximum daily demand is unavailable at the present time, therefore a factor of 1.6 was used to multiply the average daily demand to estimate the maximum daily demand projections.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	2,143	2,283	2,376
average daily demands (mgd)	0.177	0.189	0.197
maximum daily demands (mgd)	0.283	0.303	0.315
2031 population projections	2,357	3,277	3,769
average daily demands (mgd)	0.195	0.272	0.312
maximum daily demands (mgd)	0.313	0.435	0.500

Note: The population projections for other years for Ayr are shown in Appendix B.

(iv) Potential Future Supplies

The existing supply of 1.037 mgd is likely more than sufficient to meet the average daily demands projected for the year 2031. Should additional water supplies become necessary, there appears to be a good potential for further development in the overburden. The 'Ground-water Yields From Overburden' map (Figure 3.5) indicates potential yields in the Ayr area in excess of 200 gpm (0.288 mgd). As well, an OWRC ground-water survey (Pitts, 1971) indicated that high yields of satisfactory quality may be possible from the underlying bedrock.

(v) Summary

The 1979 water-supply system of Ayr consisted of two overburden wells with a combined rated capacity of 1.037 mgd. This supply appears to be more than sufficient to meet the average daily demands, based on a medium population projection, in the year 2031. There appears to be a good potential in this area for additional ground-water development, both in overburden and in the bedrock.

4.2.3 Baden - New Hamburg

(i) Location and Physical Description

The two communities of Baden and New Hamburg are discussed together in this report because their respective water-supply systems have been combined into one by a 3-mile pipeline.

New Hamburg is located on the Nith River, approximately 16 miles west-southwest of Kitchener. Baden is situated approximately 3 miles east of New Hamburg.

Overburden thickness varies from about 100 to 300 feet and is generally about 150 feet over most of the area. The overburden consists of clayey tills interbedded with extensive sand and gravel deposits. The surficial deposits over most of the area are kame and outwash sands and gravels that reach a thickness of approximately 300 feet in the Baden Hills area. The underlying bedrock is shale and dolomite of the Salina Formation, which is overlain in the New Hamburg area by dolomite of the Bass Island Formation.

(ii) Existing Supplies

The 1979 municipal water-supply systems of Baden and New Hamburg consisted of two overburden wells in Baden rated at 1.382 mgd and two overburden wells in New Hamburg rated at 0.302 mgd (Figure 4.2). The two systems are joined together with a 12-inch diameter pipeline and storage is provided by a 400,000-gallon capacity standpipe. The New Hamburg wells have not been pumped in the past

few years because of poor quality water but rather serve as a standby supply. The average and maximum daily demands on this system in 1977 were 0.409 and 0.649 mgd, respectively.

Ground water in this area is generally very hard, high in alkalinity, and often has elevated iron concentrations. Generally, wells developed in the Salina bedrock report much higher concentrations than those developed in overburden. The hardness of the overburden ground water is attributed to the presence of fragments of Salina dolomite in the till. Shallow overburden wells report better water quality than deep (overburden) wells.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population - Baden	1040	1109	1153
- New Hamburg	4535	4832	5028
- combined	5575	5941	6181
average daily demands (mgd) - combined	0.512	0.546	0.568
maximum daily demands (mgd) - combined	0.813	0.866	0.901
2031 population - Baden	1145	1591	1830
- New Hamburg	4989	6935	7977
- combined	6134	8526	9807
average daily demands (mgd) - combined	0.563	0.783	0.901
maximum daily demands (mgd) - combined	0.894	1.243	1.430

Note 1. Seperate demand values for Baden and New Hamburg are not available at this time.

Note 2. Population projections for other years for New Hamburg are listed in Appendix B, and for Baden in Appendix C.

(iv) <u>Potential Future Supplies</u>

The existing supply of 1.684 mgd is sufficient to satisfy the average daily demand projected for the year 2031. If additional water supplies are required, there appears to be a good potential for well development throughout the Baden-New Hamburg area. The 'Ground-Water Yields from Bedrock' map (Figure 3.4) identifies bedrock yield in this area as being up to 200 gpm (0.288 mgd). Overburden in the area is described on the 'Ground-Water Yields From Overburden' map (Figure 3.5) as having potential yields of up to 100 gpm, with a small area to the south of Baden (corresponding in location to the Baden Hills) having a potential to yield in excess of 200 gpm per well.

Ground-water surveys for Baden (Sobanski, 1968) and New Hamburg (Hore 1960) carried out by the OWRC recommended that in spite of the higher potential yields from bedrock, future municipal supplies should be developed in the overburden aquifers near Baden because of their better chemical water quality. The 1968 survey identified two sand and gravel aquifers in the Baden area. The upper aquifer is found at a depth of 60 to 100 feet, with a thickness of 10 to 30 feet. Most of the overburden wells in this area, including the Baden municipal wells, are developed in this formation. The extent and depth of the lower aquifer are not well known because few wells in the area extend beyond the upper formation. Generally, these lower sands and gravels are found near the overburden-bedrock interface and are seperated from the upper formation by 80 to 120 feet of relatively impervious clay and/or till. The similarity in the quality of water from this formation and from bedrock suggests that the two may be hydraulically connected in some places. This basal formation has a maximum known thickness of about 50 feet in the vicinity of well 2443 (Figure 4.2).

In accordance with the recommendations of the previous surveys, an area northeast of Baden appears to have a good potential for future development of municipal supplies. In this area, about 300 feet of clay and till overlie Salina bedrock. An MOE test well (3735) drilled in 1972 in the area reported an upper sand and gravel formation extending from 75 to 115 feet from the surface, and a lower

sand and gravel formation extending from 155 to 163 feet. Near-by well 2421, which is developed in the lower formation, has a specific capacity of 10 gpm/ft and an available drawdown of about 90 feet. The remaining wells in the recommended test-drilling area are developed in the upper formation and have specific capacities ranging from 2.0 to 25.7 gpm/ft. Well 4429 is an exception because it does not show any permeable overburden deposits and is developed in bedrock.

It is important to note that in the event that the upper aquifer should be further developed as a source of municipal supply for Baden-New Hamburg, the potential for interference with existing wells should be investigated fully. Kitchener-Waterloo municipal wells K-50, K-51 and K-52 are within 3 miles of the recommended test-drilling area and are developed in a formation similiar in composition and elevation to the upper aquifer in the recommended area. If these formations are continuous, the effect on existing wells by additional municipal water takings could be significant.

(v) Summary

The 1979 water-supply system of Baden-New Hamburg consisted of two overburden wells in Baden, connected by pipeline to two overburden wells in New Hamburg. The combined rated capacity of 1.684 mgd should be sufficient to satisfy the average daily demands projected for the year 2031. A potential test-drilling area for additional municipal supplies has been identified. The development of the upper aquifer within this recommended area could result in interference problems with near-by Kitchener-Waterloo municipal wells and private wells in the area.

4.2.4 Caledonia

(i) Location and Physical Setting

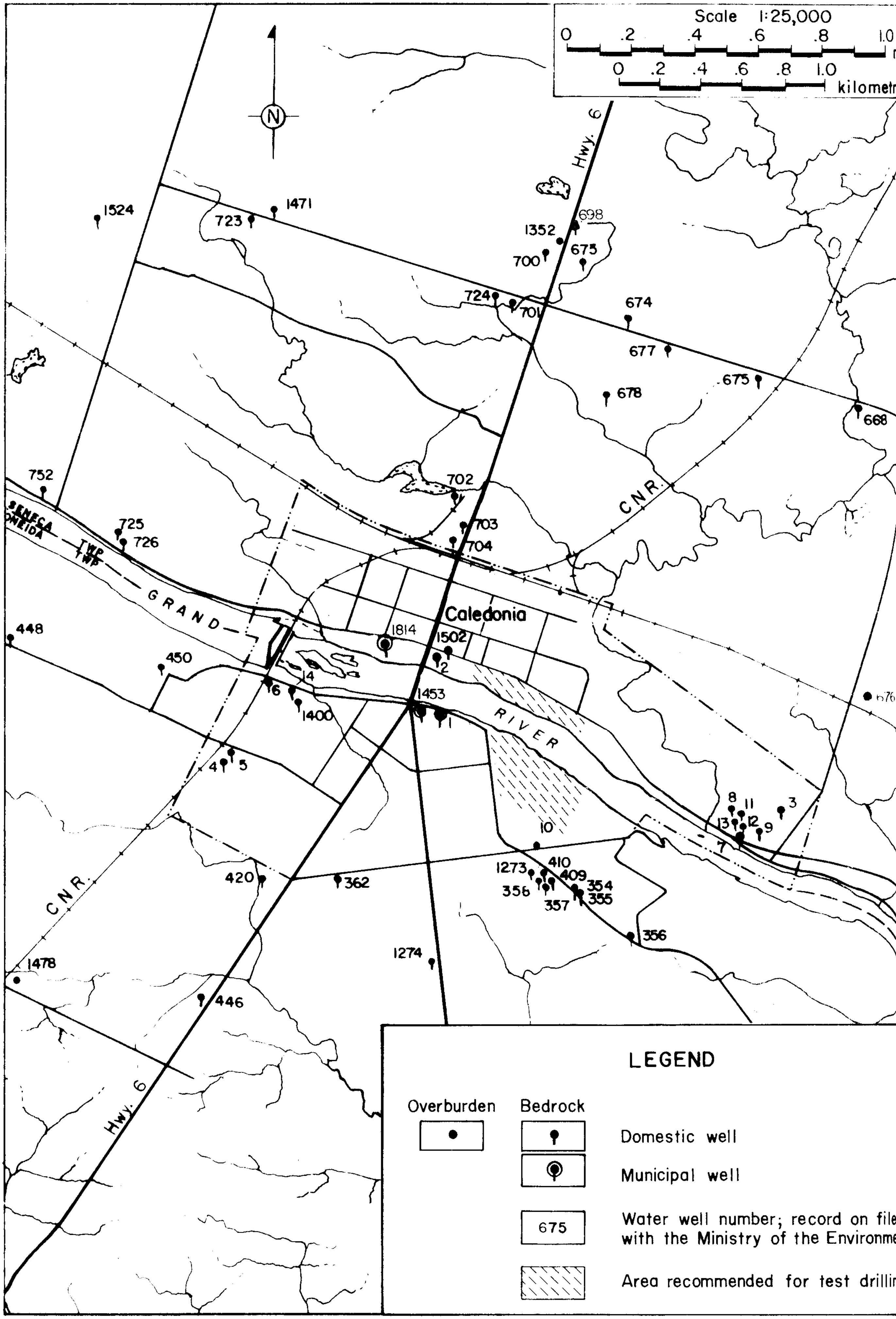
Caledonia is located at the junction of Highway No. 6 and the Grand River, approximately 19 miles east of Brantford and 42 miles southeast of Kitchener.

Overburden thickness in the area varies from less than 10 to 90 feet, and consists primarily of clays that commonly extend to bedrock. In some locations the surficial clays are underlain by till or sand and/or gravel deposits up to 32 feet thick. East of Caledonia the clay is overlain by shallow, fine sandy loam soil, which is generally less than 5 feet thick. The underlying bedrock around Caledonia consists of shales of the Salina Formation, which overlie dolomites of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system of Caledonia consisted of five bedrock wells, with a combined rated capacity of 2.560 mgd. Well 1453 (Figure 4.3) is pumped only during the summer months, and well 1 represents three wells (housed within one building), one of which is on stand-by. The other two wells at number 1, and well 1814, are pumped throughout most of the year. The 1977 average and maximum daily demands on this system were 0.323 and 0.645 mgd, respectively.

The chemical quality of ground water in the area is generally unsatisfactory for public supplies. The municipal water is extremely hard (1440 mg/L) and exceeds recommended criteria for iron, sulphates and total dissolved solids.



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Bedrock
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Municipal well
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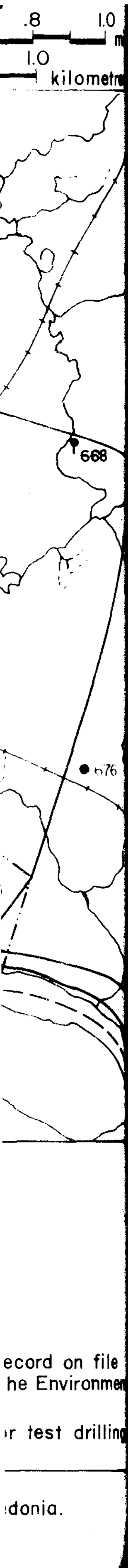
Water well number; record on file with the Ministry of the Environment
Area recommended for test drilling



(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	6,500	9,750	13,000
average daily demands (mgd)	0.572	0.858	1.144
maximum daily demands (mgd)	1.144	1.716	2.288
2031 population projections	13,000	15,824	18,648
average daily demands (mgd)	1.144	1.392	1.641
maximum daily demands (mgd)	2.288	2.785	3.282

Note: The population projections for other years for Caledonia are shown in Appendix B.

(iv) Potential Future Supplies

The existing municipal water supply of 2.560 mgd will likely be more than sufficient to meet the average daily demands in the year 2031. Should the development of additional supplies become necessary, two areas recommended for test drilling are indicated on Figure 4.3. A ground-water survey for Caledonia (Andrijiw, 1973) recommended that bedrock penetration should not exceed about 15 feet due to the likely deterioration of water quality with depth.

(v) Summary

The 1979 water-supply system of Caledonia consisted of five bedrock wells with a combined rated capacity of 2.560 mgd. This supply will likely be sufficient to meet the average daily demand, based on a medium population projection, for the year 2031. Two areas recommended for test-drilling for additional municipal supplies have been identified.

4.2.5 Dundalk

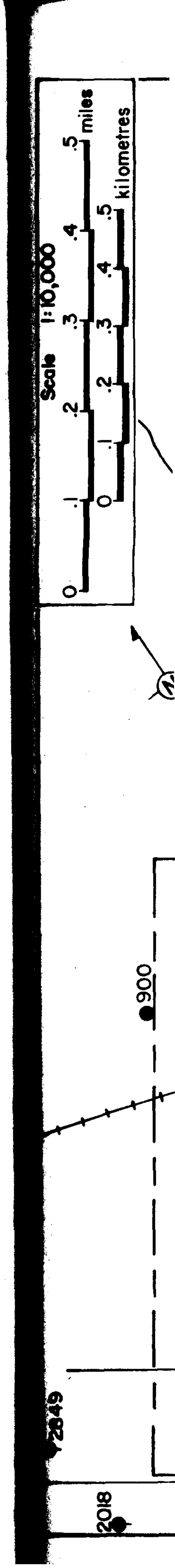
(i) Location and Physical Setting

Dundalk is located in the northern-most portion of the basin at the headwaters of the Grand River, approximately 60 miles north of Kitchener.

Overburden thickness in the area varies from about 50 to 100 feet, and consists mainly of a sandy, silt till overlain by shallow outwash gravels at the surface. The underlying bedrock is dolomite of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system of Dundalk consisted of 3 bedrock wells with a combined rated capacity of 0.432 mgd (Figure 4.4). These wells are from 201 to 285 feet deep, with a bedrock penetration of 98 to 193 feet. Water is encountered at various depths between 104 and 248 feet. Water quality analyses of municipal wells 897 and 898 indicate that water from these depths is generally of good quality. The 1977 average and maximum daily demands on this system were .113 mgd and .201 mgd, respectively.



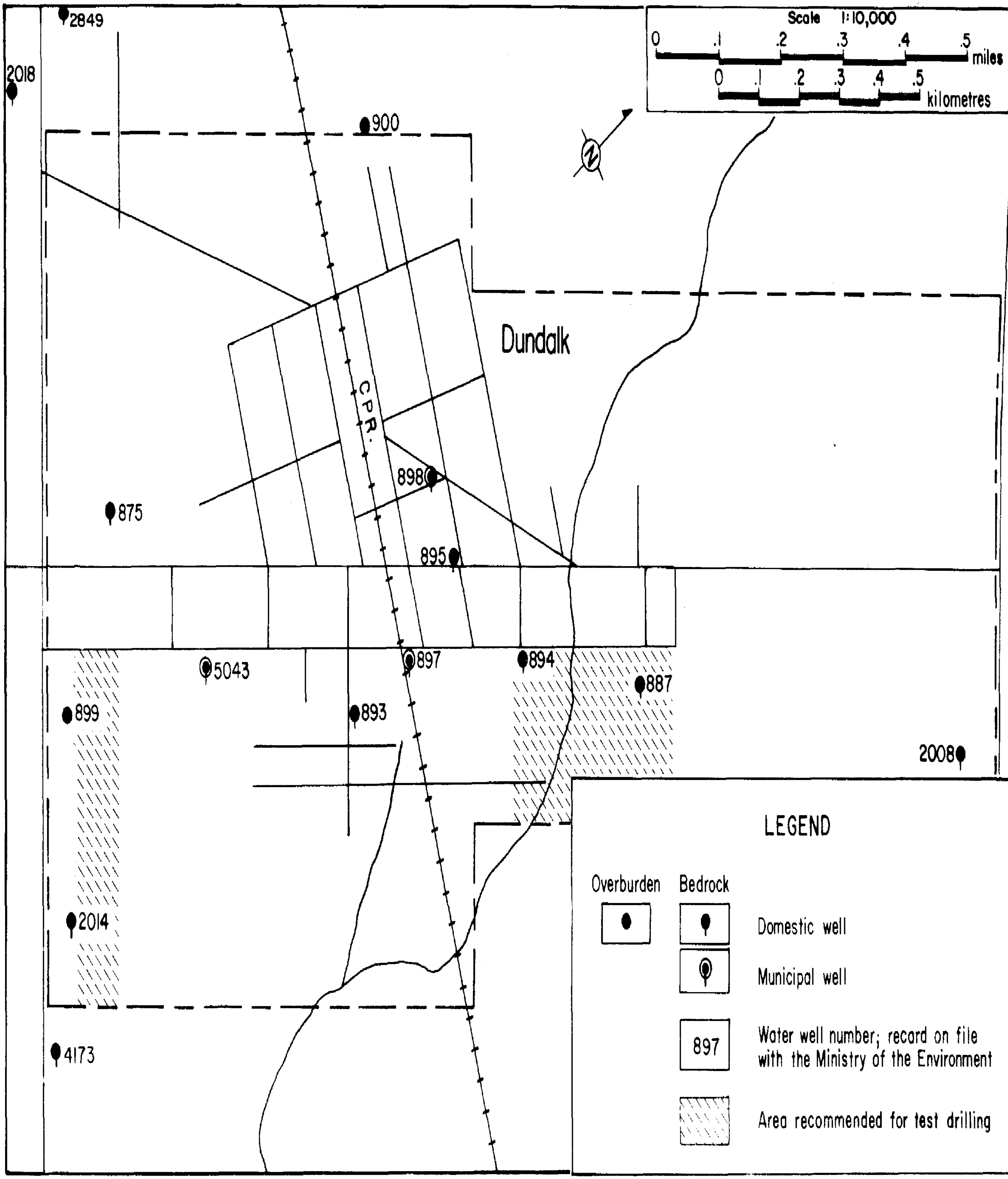


Figure 4.4. Locations of water wells and recommended test-drilling areas for Dundalk.

(iii) Future Demand

Average daily and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	1561	1854	2094
average daily demands (mgd)	0.156	0.185	0.209
maximum daily demands (mgd)	0.278	0.330	0.372
2031 population projections	2238	3350	4394
average daily demands (mgd)	0.224	0.335	0.439
maximum daily demands (mgd)	0.398	0.596	0.782

Note: Population projections for other years for Dundalk are shown in Appendix B.

(iv) Potential Future Supplies

The 1979 municipal water supply of 0.432 mgd will likely be sufficient to meet the average daily demand in the year 2031 (based on a medium population projection).

The "Ground-Water Yields From Bedrock" map (Figure 3.4) indicates the probable yield from bedrock in this area to be in excess of 200 gpm (0.288 mgd), and two potential test-drilling areas for the development of new wells have been identified. The eastern area has about 100 feet of overburden overlying bedrock. The two existing wells in this area (887 and 894) penetrate bedrock by about 50 feet and have specific capacities of 2.0 gpm/ft and 2.5 gpm/ft, with approximately 130 and 140 feet of available drawdown, respectively. The western area has about 75 feet of overburden over bedrock. Well 2014 in the area has a specific capacity of 6.0 gpm/ft and an available drawdown of about 70 feet. Bedrock penetration is 19 feet.

(v) Summary

The 1979 municipal water-supply system of Dundalk consisted of 3 bedrock wells with a combined rated capacity of 0.432 mgd. This supply will likely be sufficient to meet the average daily demand in the year 2031, based on a medium population projection. Probable bedrock yield in the area is likely to be in excess of 0.288 mgd. Two areas presently accessable for test drilling have been identified.

4.2.6 Elmira - St. Jacobs

(i) Location and Physical Setting

The two communities of Elmira and St. Jacobs are discussed together in this report because their respective water-supply systems have been combined into one by means of a 3.5-mile long pipeline.

St. Jacobs is located on the Conestogo River approximately 10 miles north of Kitchener. Elmira is located 3 miles north of St. Jacobs.

Overburden thickness varies from about 80 to 200 feet, and is generally about 150 feet throughout most of the area. The overburden consists of clay and till interbedded with extensive, but generally unconnected, sand and gravel deposits. The latter materials are most extensive as basal deposits of fine to coarse sands and gravels, with thickness of up to 80 feet. Near Elmira there are also thick sand and gravel deposits reported at various depths in the overburden, as well as surficial deposits of kame sands and gravels. The underlying bedrock consists of limestone, dolomite and shale of the Salina Formation.

(ii) Existing Supplies

The 1979 municipal water-supply systems of Elmira and St. Jacobs consisted of six overburden wells in Elmira, with a total rated capacity of 4.46 mgd, and two bedrock wells in St. Jacobs rated at 0.15 mgd (Figure 4.2). The two systems are connected with a 3.5-mile long, 12-inch diameter pipeline. The total rated capacity

of the combined system is 4.61 mgd. The 1977 demand on this system was 1.68 mgd and 2.81 mgd for average and maximum daily consumption, respectively.

The chemical water quality of overburden and bedrock ground waters in the area is similar, probably because most of the high-yield overburden wells are developed in permeable deposits that overlie, and are probably hydraulically connected to the bedrock. Ground water in the area is usually hard to very hard (about 400 mg/L), iron content ranges from 0.4 to 2.1 mg/L and some elevated sulphate concentrations have been reported.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections			
- Elmira	8,701	9,272	9,648
- St. Jacobs	1,048	1,117	1,162
- combined	9,749	10,389	10,810
average daily demands (mgd)			
- Elmira	1.955	2.083	2.168
- St. Jacobs	0.123	0.131	0.136
- combined	2.078	2.214	2.304
maximum daily demands (mgd)			
- Elmira	3.277	3.492	3.634
- St. Jacobs	0.197	0.210	0.218
- combined	3.474	3.702	3.852
2031 population projections			
- Elmira	9,574	13,308	15,307
- St. Jacobs	1,153	1,603	1,844
- combined	10,727	14,911	17,151
average daily demands (mgd)			
- Elmira	2.151	2.990	3.439
- St. Jacobs	0.135	0.188	0.216
- combined	2.286	3.178	3.655
maximum daily demands (mgd)			
- Elmira	3.606	5.013	5.766
- St. Jacobs	0.217	0.301	0.347
- combined	3.823	5.314	6.113

NOTE: The population projections for other years for Elmira are shown in Appendix B, and for St. Jacobs in Appendix C.

(iv) Potential Future Supplies

The total rated capacity of the 1979 system (4.61 mgd) appears to be sufficient to satisfy the average daily demands in the year 2031. It is likely that additional ground-water supplies could be developed in the Elmira-St. Jacobs area.

The perennial safe yields of wells E2-E8 and E7-E9 have been assessed at 3.0 and 2.4 mgd, respectively, in past ground-water surveys for Elmira (Dixon, 1961) and St. Jacobs (International Water Supply, 1976). This indicates that an additional 0.79 mgd could be extracted from the aquifers presently being pumped. The 1976 survey recommended that additional supplies be developed near test well 4515, about one mile south of well E7 along Highway No. 85.

An area recommended for test drilling for additional municipal supplies is shown in Figure 4.2. This area is adjacent to, and may be part of, a zone described on the 'Ground-Water Yields from Overburden' map (Figure 3.5) as having a potential for well yields in excess of 200 gpm. The overburden in this area is mainly clay and till 100 to 250 feet thick, with isolated sand and gravel lenses near the surface and/or at depths of 60 to 80 feet. Although it is not known if the basal sands and gravels in this area are continuous with those of the E7-E9 aquifer, they are found at approximately the same elevation. Existing wells finished in this formation have specific capacities varying from 3.0 to 13.00 gpm/ft.

There appears to be a potential for additional municipal supplies through artificial recharge approximately three miles north of Elmira. However, this recharge is contingent on the construction of the West Montrose reservoir and will be discussed in a later section dealing with water-supply options for Kitchener-Waterloo.

(v) Summary

The 1979 water-supply system of Elmira-St. Jacobs consisted of six overburden wells in Elmira connected via a pipeline to two bedrock wells in St. Jacobs. The combined rated capacity of 4.61 mgd will likely be sufficient to meet the estimated average daily demand in the year 2031. An additional 0.79 mgd is estimated to be available from the existing well fields. An area containing wells with high specific capacities adjacent to the E7-E9 well field is recommended for future test drilling for addition municipal supplies.

4.2.7. Maryhill

(i) Location and Physical Setting

The community of Maryhill is located at the junction of county roads 25 and 39, approximately 5 miles west of Guelph and 9 miles north-east of Kitchener.

Overburden thickness in the area is about 100 feet, and consists of a stoney, sandy silt till overlain by interbedded layers of clay, sand and gravel. To the north and west of Maryhill, this sequence is overlain primarily by clay till that is a remnant of the Breslau Moraine. To the east of Maryhill and corresponding generally to the Hopewell Creek valley are surficial deposits of outwash gravels of undetermined depth. The underlying bedrock in the area consists of dolomites of the Guelph Formations.

(ii) Existing Supplies

In 1979 only the Maryhill sub-division on Isley Drive was serviced by a water-supply system operated by the Regional Municipality of Waterloo. The serviced population in 1976 was 150, roughly one-third of the total population of Maryhill. The existing water-supply system in 1979 consisted of one overburden well and one bedrock well, with a combined rated capacity of 0.072 mgd (Figure 4.2). The 1977 average daily demand on this system was 0.008 mgd. There is no information available concerning maximum daily demand.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	514	548	570
average daily demands (mgd)	0.029	0.031	0.032
maximum daily demands (mgd)	0.047	0.050	0.052
2031 population projections	566	786	904
average daily demands (mgd)	0.032	0.045	0.052
maximum daily demands (mgd)	0.052	0.072	0.082

NOTE: The population projections for other years for Maryhill are shown in Appendix C.

(iv) Potential Future Supplies

The future demand projections were calculated with the assumption that averge daily demands of the total community will be the same as those of the presently serviced sub-division - that is, 57 gallons per day per capita. Maximum daily demands were calculated as being 1.6 times the average daily demands. Although the existing municipal supply system is presently servicing only about one-third of the community of Maryhill, the rated capacity of this system will likely be sufficient to meet the average daily demands of the entire community in 2031. If municipal water supply is to be extended to the rest of the community, it is likely that additional wells need to be added to the existing system. A sub-division well presently being developed in the

northern part of this community could become incorporated into the municipal system if the yield and water quality prove to be satisfactory. As well, additional ground-water supplies could be developed both in the overburden sand and gravel deposits, and in the Guelph Formation bedrock.

(v) Summary

The community of Maryhill is dependant largely on private domestic wells for its water supply. A sub-division on Isley Drive is serviced by one overburden well and one bedrock well operated by the Regional Municipality of Waterloo. The combined rated capacity of these wells (0.072 mgd) will likely be sufficient to meet the average daily demands of the entire community in 2031. There is a good potential for additional ground-water development in this area.

4.2.8 Milverton

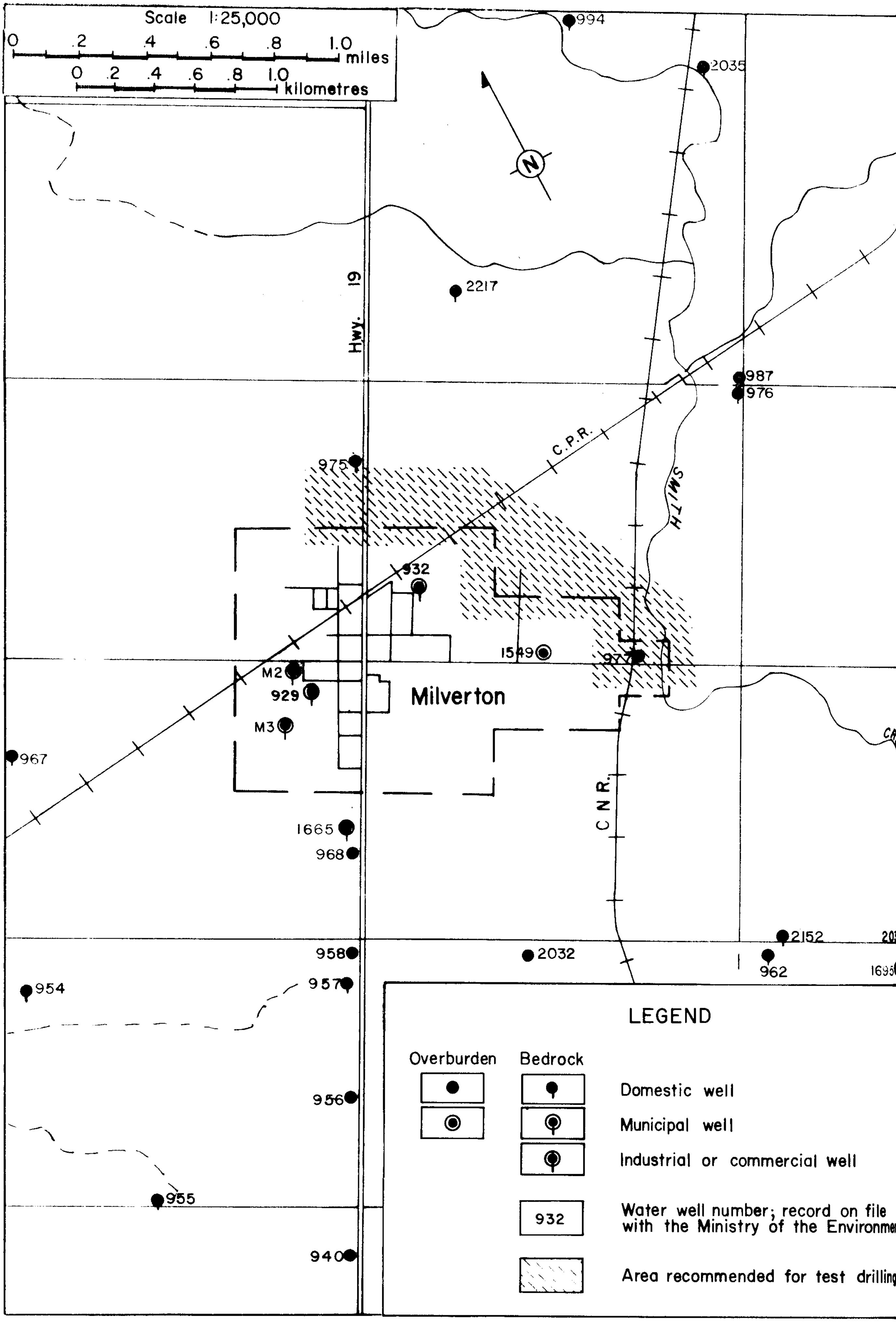
(i) Location and Physical Setting

The Village of Milverton is located at the western edge of the Grand River basin, approximately 26 miles west of Kitchener.

Overburden thickness is about 150 feet over most of the area, increasing to more than 300 feet in the buried bedrock valley to the south of Milverton. The overburden consists of till and clay interbedded with sands and gravels. The bedrock consists of limestone and dolomite of the Bois Blanc Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system for Milverton consisted of one bedrock and one overburden well, with a combined rated capacity of 0.315 mgd, and two standby bedrock wells (M2 and M3 - presently unequipped), with a combined rated capacity of 0.252 mgd (Figure 4.5).



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The two standby wells were the original source of Milverton's municipal supply, but because they yielded very hard water, they were replaced with wells 932 and 1549. The water from the new wells contains some sulphur and is treated with silicate to reduce the high iron content.

The average daily demand on this system in 1977 was 0.129 mgd. Information concerning maximum daily demands is unavailable at the present time. For future demand projections, maximum daily demands were calculated as being 1.6 times the average daily demands.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high projections, are as follows:

	low	medium	high
2001 population projections	1516	2057	2935
average daily demands (mdg)	0.139	0.189	0.270
maximum daily demands (mgd)	0.222	0.302	0.432
2031 population projections	1664	3257	7125
average daily demands (mgd)	0.153	0.299	0.655
maximum daily demands (mgd)	0.245	0.478	1.043

Note: The population projections for other years for Milverton are shown in Appendix B.

(iv) Potential Future Supplies

The 1979 supply of 0.315 mgd should be sufficient to satisfy the average daily demand, based on a medium population projection, in the year 2031.

An obvious source of additional water supplies is the two standby wells that were used for municipal supply in the past. These wells experienced no water quality problems beyond hardness, and if re-activated, could increase the existing supply by 0.252 mgd. As well, there appears to be a potential in the Milverton area for the development of additional ground-water supplies in the area shown on Figure 4.5.

The "Ground-Water Yields From Bedrock" map (Figure 3.4) identifies the bedrock yields around Milverton to be in excess of 200 gpm (0.288 mgd). A survey of ground-water resources in the Milverton area (Wilson, 1962) determined that relatively large yields from bedrock were possible anywhere in the Milverton area, and that, although sand and gravel aquifers appear to be discontinuous and limited in extent, it may be possible to develop high-yield overburden wells at some locations. Water obtained from overburden was found to be superior in chemical quality to bedrock ground water, and water obtained from the top 40 feet of the Bois Blanc Formation was of better quality than that obtained at greater depths.

The area recommended for test drilling is presently accessible for new well development, and the proximity of the two active municipal wells to this area indicates that potentially high yields are possible from overburden and bedrock. Overburden in the area is approximately 140 feet thick, with sand and gravel deposits reported at depths of 5, 25 and 95 feet. In accordance with findings of the 1962 survey, it is recommended that water-bearing deposits in the overburden be tested first. If no adequate supplies can be obtained from the overburden, the top 40 feet of the Bois Blanc Formation should be explored.

(v) Summary

The 1979 municipal water-supply of Milverton consisted of two wells, with a combined rated capacity of 0.315 mgd. This supply will likely be sufficient to meet the average daily demand, based on a medium population projection, in the year 2031. Additional supplies can be obtained from two un-equipped, stand-by wells rated at 0.252 mgd. Overburden and bedrock in the area have potential for future development, and an area recommended for test drilling has been identified.

4.2.9 Paris

(i) Location and Physical Setting

The Town of Paris is located at the confluence of the Grand and Nith rivers, approximately 19 miles south of Kitchener.

Overburden thickness in the area varies from about 10 to 200 feet, with the thickest deposits located to the north and west of Paris. An average overburden thickness throughout the area is about 100 feet, consisting mainly of glaciofluvial sands and gravels at the surface, underlain by a sequence of silt and till interbedded with lenses of sand and gravel. The surficial gravels to the north of Paris are up to 60 feet thick. The underlying bedrock in this area consists of dolomite and shale of the Salina Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system for Paris consisted of a shallow ground-water collector system which was upgraded in late 1979 to a capacity of 1.728 mgd, and two overburden wells rated at 1.152 mgd (Figure 4.6). The wells are generally used only to supplement supplies during peak demand periods in the summer because they yield very hard water (400 - 500 mg/L). The 1977 average and maximum daily demands on this system were 1.064 and 1.630 mgd, respectively.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low medium and high population projections, are as follows:.

	low	medium	high
2001 population projections	8,000	8,800	9,484
average daily demands (mgd)	1.024	1.126	1.214
maximum daily demands (mgd)	1.648	1.813	1.954
2031 population projections	8,609	10,921	13,168
average daily demands (mgd)	1.102	1.398	1.685
maximum daily demands (mgd)	1.773	2.249	2.713

Note: The population projections for other years for Paris are shown in Appendix B.

(iv) Potential Future Supplies

The 1979 water supply of 2.88 mgd will likely be more than sufficient to meet the estimated average daily demand projected for the year 2031. In the event that additional municipal supplies will be required, an area recommended for test drilling is outlined in Figure 4.6. Well logs of existing wells within this recommended area indicate that there may be two overburden aquifers suitable for development. Most of the overburden wells near the Grand River, including the two Paris municipal wells, are developed in basal sands and gravels at depths from 100 to 175 feet. These deposits are generally less than 20 feet thick, except at locations alongside the river. The specific capacities of wells developed in this formation range from 1.43 gpm/ft (well 1923) to 76.0 gpm/ft (well 1153). Most of the overburden wells away from the Grand River are developed in an upper sand and gravel formation that is up to 80

feet thick found at 50 to 100 feet from the surface. Specific capacities of wells in this formation vary from 0.41 gpm/ft (1022) to 13.33 gpm/ft (978). It is not known whether these sand and gravel formations represent one continuous aquifer from St. George to the Grand River, or a number of smaller aquifers of varying thicknesses.

(v) Summary

The 1979 municipal water-supply system of Paris consisted of two overburden wells and a shallow ground-water collector system with a combined rated capacity of 2.88 mgd. This supply will likely be greater than the average daily demand projected for the year 2031. An area recommended for future test drilling appears to contain two overburden aquifers - a basal formation near the Grand River and an upper formation extending eastward to St. George.

4.2.10 Plattsville

(i) Location and Physical Setting

The community of Plattsville is located at the junction of Highway No. 97 and the Nith River, approximately 15 miles south - southwest of Kitchener (Figure 4.2).

Overburden in the area is about 150 feet thick and consists mainly of kame and outwash sands and gravels, interbedded with clay, silt and clay till. The underlying bedrock consists of shale and dolomite of the Salina Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system for Plattsville consisted of two overburden wells, with a combined rated capacity of 0.374 mgd. This system was installed in late 1979, and at the end of 1979 only one well was equipped. The municipal water supply has high sulphate concentrations and is very hard. There is no information available on water uses within this community.

(iii) Future Demand

For the future demand projections, average and maximum daily demands were assumed to be 100 and 160 gallons per day per capita, respectively. Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections are, as follows:

	low	medium	high
2001 population projections	564	639	723
average daily demands (mgd)	0.056	0.064	0.072
maximum daily demands (mgd)	0.090	0.102	0.116
2031 population projections	655	861	1,129
average daily demands (mgd)	0.065	0.086	0.113
maximum daily demands (mgd)	0.105	0.138	0.181

NOTE: Population projections for other years for Plattsville are shown in Appendix C.

(iv) Potential Future Supplies

The existing municipal water supply of 0.374 mgd is more than three times the projected average daily demand in the year 2031. In the event that additional municipal supplies become necessary, there appears to be a potential for future well development in the buried sand and gravel layers throughout this area. A report on test drilling and well construction in Plattsville (Geo-Environ., 1979) stated that although water-bearing sands and gravels were encountered at various depths, deposits generally nearer the surface (about 40 feet) are likely to yield better quality water than those close to bedrock. Accordingly, it is recommended that future test drilling for municipal supplies in this area be directed to the upper water-bearing sands and gravels.

(v) Summary

The community of Plattsville has had a municipal water supply system installed in 1979, which consists of two overburden wells having a combined rated capacity of 0.374 mgd. Only one well was equipped in 1979. This supply will likely exceed projected demand in the year 2031. Additional supplies may be developed in overburden in this area. Better quality water is available from shallower overburden deposits than from deeper aquifers.

4.2.11 Rockwood

(i) Location and Physical Setting

The community of Rockwood is located at the junction of Highway No. 7 and the Eramosa River, approximately 7 miles northeast of Guelph and 20 miles northeast of Kitchener (Figure 4.2).

Overburden in the area consists mainly of a stoney clay till and surficial outwash sand and gravel deposits, with a total thickness of about 25 feet. The underlying bedrock consists of dolomites of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system for Rockwood consisted of two bedrock wells, with a combined rated capacity of 0.864 mgd. This system began operation in December 1978. Generally, ground water in this area is hard, but otherwise of good quality for municipal purposes; high iron concentrations can occur locally.

The 1979 average daily demand on this system was 56 gallons per day per person. There is no information available regarding maximum daily demands.

In the absence of existing data, maximum daily demands were considered to be 1.6 times the average daily demands.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	1,420	1,817	2,208
average daily demands (mgd)	0.079	0.102	0.124
maximum daily demands (mgd)	0.127	0.163	0.198
2031 population projections	1,914	3,290	5,057
average daily demands (mgd)	0.107	0.184	0.283
maximum daily demands (mgd)	0.171	0.295	0.453

Note: The population projections for other years for Rockwood are shown in Appendix B.

(iv) Potential Future Supplies

The existing supply of 0.864 seems more than sufficient to meet the average daily demand projected for 2031. Should additional water supplies become necessary, bedrock in the area of Rockwood may be expected to yield large quantities of good-quality water. The 'Ground-Water Yields From Bedrock' map (Figure 3.4) indicates likely yields from bedrock in this area to be in excess of 200 gpm (0.288 mgd).

(v) Summary

The 1979 municipal water-supply system of Rockwood consisted of two bedrock wells with a combined rated capacity of 0.864 mgd. Bedrock ground water in this area is hard but generally of good chemical quality. The present supply will likely be more than sufficient to meet expected demand in 2031. Additional supplies in excess of 200 gpm per well may be developed from bedrock.

4.2.12. St. George

(i) Location and Physical Setting

The community of St. George is located on Highway No. 5 east of Highway No. 24, approximately 8 miles east of Paris and 18 miles southeast of Kitchener (Figure 4.6).

Overburden thickness varies from 100 to 200 feet and consists mainly of till interbedded with sand, silt and clay at depth. The underlying bedrock is dolomite of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system consisted of one flowing overburden well rated at 1.8 mgd. The 1977 average and maximum daily demands on this system were 0.15 and 0.23 mgd, respectively. These values appear high on a per capita basis because of the very large water usage by Malcolm Condensing in their dairy products processing plant.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	1,659	1,766	2,000
average daily demands (mgd)	0.267	0.284	0.322
maximum daily demands (mgd)	0.409	0.436	0.494
2031 population projections	3,317	3,803	4,997
average daily demands (mgd)	0.534	0.612	0.804
maximum daily demands (mgd)	0.819	0.939	1.234

Note: Population projections for other years for St. George are shown in Appendix C.

(iv) Potential Future Supplies

The existing supply of 1.8 mgd will likely be more than sufficient to meet the average daily demand in the year 2031. In the event that additional municipal supplies are required, the area immediately around and to the west of St. George is recommended for test drilling (Figure 4.6). Existing wells in this area have reported water- bearing sands and gravels of various thicknesses at depths from 22 to 70 feet. Specific capacities of these wells range from 0.5 gpm/ft (well 1002) to 9.5 gpm/ft (well 955). There appears to be a good potential for additional municipal well development in the area.

(v) Summary

The 1979 municipal water-supply system of St. George consisted of one flowing overburden well rated at 1.8 mgd. This supply will likely exceed the average daily demand projected for the year 2031. There appears to be a good potential for additional municipal supply development in overburden in the St. George area. A recommended test-drilling area has been identified.

4.3. Communities with Insufficient Supplies to Meet Demands After 2031

The communities whose existing municipal water supplies will likely not be sufficient to meet the average daily demands, based on a medium population projection in the year 2031, are:

Cambridge

Elora

Fergus

Guelph

An evaluation of the existing and future water supply/demand situation for each community follows.

4.3.1 Cambridge

(i) Location and Physical Setting

The City of Cambridge is an amalgamation of the previously seperate towns of Galt, Hespeler and Preston. Cambridge is located at the confluence of the Grand and Speed rivers, approximately 6 miles southeast of Kitchener (Figure 4.2).

Overburden thickness varies from 0 along some sections of the Grand and Speed rivers, to about 180 feet near Glen Morris to the south of Cambridge, but is generally from 50 to 100 feet over most of the area. The overburden consists primarily of till interbedded with silt, clay, sand and gravel layers 5 to 40 feet thick. Surficial outwash gravels of up to 50 feet in thickness overlie till to the west of Cambridge and along the Grand and Speed rivers.

(ii) Existing Supplies

The 1979 municipal water-supply system of Cambridge consisted of 22 bedrock wells and 3 overburden wells, with a combined rated capacity of 16.99 mgd (Table 4.2). The 1977 average and maximum daily demands on this system were 8.577 and 12.866 mgd, respectively.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	116,815	124,474	129,525
average daily demands (mgd)	14.02	14.94	15.54
maximum daily demands (mgd)	21.03	22.40	23.31
2031 population projections	128,524	178,662	205,495
average daily demands (mgd)	15.42	21.44	24.66
maximum daily demands (mgd)	23.13	32.16	36.99

Note: The population projections for other years for Cambridge are shown in Appendix B.

(iv) Potential Future Supplies

The 1979 municipal water supply of 16.99 mgd appears to be sufficient to meet the average daily demand projected for the year 2001. In order to meet the average daily demand in the year 2031, based on a medium population projection, an additional supply of 4.45 mgd will need to be developed.

TABLE 4.2 Municipal Water-Supply System for Cambridge, 1979

Well No.	Location	Rated Capacity (mgd)
G1	Middleton St.	2.088
G2	Middleton St.	3.456
G3*	Middleton St.	2.736
G4*	Blair Rd.	0.367
G5*	Pinebush Rd.	0.604
G6*	Cowan	0.604
G7*	Clyde 1	0.547
G8^{8}	Clyde 2	0.316
G9*	Elgin	0.604
G10*	Shades Mills	1.008
G12*	Branchton	0.050
P2	Riverside Park)	
P5	Riverside Park)	0.720
Springs	Riverside Park)	
P4*	Coronation Blvd.	0.072
P6*	Dunbar R.	0.518
P7*	Dunbar R.	0.100
P9*	Rahman Corners	0.518
P10*	Pinebush Rd. East	0.417
P11*	Speedvale Farm	0.604
H3*	Winston Blvd.	0.288
H4*	Hungerford Rd.	0.172
H5*	Guelph Ave.	0.381
H1	Milling Rd.	0.158
H2	Milling Rd.	0.230
H6*	Sheffied St.	0.432
	Total	16.99

* denotes well pumped directly into system

The Regional Municipality of Waterloo has carried out an extensive study of ground-water resources in the Cambridge area during 1978 and 1979, and the study is to be completed by further investigations in 1980. Although the final results of the whole study will not be available until late in 1980, preliminary results (Morrison Beatty, 1978) indicate that a total yield of about 8.0 mgd could be developed from eleven sites investigated to the end of 1979.

To facilitate the comparison of costs of various water-supply options for Cambridge, a project evaluation of the development of six of these sites was carried out and is included in Appendix D. A general estimate of 6.120 mgd from these sites was used in the project evaluation. A more precise evaluation of the potential of these and other sites, based on pumping tests, will be given in the final report by Morrison Beatty. If the total potential of the final sites is equal to or greater than the 6.120 mgd estimate used for costing purposes, the average daily demand projected for 2031 might be satisfied.

(v) Summary

The 1979 municipal water-supply system of Cambridge consisted of 22 bedrock and 3 overburden wells, with a combined rated capacity of 16.99 mgd. This capacity will likely be sufficient to satisfy the average daily demand projected for the year 2001. However, an additional supply of about 4.45 mgd would be needed to meet the average daily demand, based on a medium population projection, after the year 2031. A three year investigation of ground-water resources in the Cambridge area has been undertaken by the Regional Municipality of Waterloo, and the final results will be available late in 1980. Preliminary investigations indicate that a potential yield of about 8.0 mgd may be available from eleven identified sites.

4.3.2 Elora

(i) Location and Physical Setting

The Village of Elora is located along the Grand River, approximately 3 miles southwest of Fergus, and about 19 miles northeast of Kitchener (Figure 4.7).

Overburden thickness in the area varies from 0 to 50 feet on the south and east side of the river, and from 0 to almost 100 feet on the north and west side. It consists primarily of clay till with localized surficial kame and outwash sand and gravel deposits. The underlying bedrock consists of dolomites of the Guelph Formation. A well-defined, narrow buried bedrock valley that runs through Fergus extends into this area along a northeast - southwest axis located south of Elora (Figure 4.7). The overburden within this buried valley is as thick as 215 feet in the vicinity of test hole 7047.

(ii) Existing Supplies

The 1979 municipal water-supply system of Elora consisted of two bedrock wells, with a combined rated capacity of 0.668 mgd. The quality of ground water around Elora is generally good, although moderately to very hard. The 1977 average and maximum daily demands on this system were 0.247 and 0.355 mgd, respectively.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	4,588	5,355	6,227
average daily demands (mgd)	0.458	0.536	0.623
maximum daily demands (mgd)	0.659	0.768	0.893
2031 population projections	7,722	10,844	15,114
average daily demands (mgd)	0.722	1.084	1.511
maximum daily demands (mgd)	1.108	1.556	2.169

Note: The population projections for other years for Elora are shown in Appendix B.

The 1979 supply of 0.668 mgd appears to be sufficient to meet the average daily demand projected for the year 2001. An additional supply of about 0.416 mgd will be required to meet the average daily demand, based on a medium population projection, in the year 2031.

(iv) Potential Future Supplies

The 'Ground-Water Yields from Bedrock' map (Figure 3.4) indicates the potential bedrock yield in the Elora area to be in excess of 200 gpm (0.288 mgd) per well. However, because of the highly variable nature of bedrock fracturing and solution cavities in this area, significantly different yields are possible even between neighbouring wells.

An area recommended for test drilling for additional municipal supplies is the buried bedrock valley that runs through the southeastern section of Elora. An MOE test hole (7047) drilled in 1979 encountered 15 feet of saturated coarse sand and gravel at depths from 191 to 206 feet in the buried valley. The geological log of this test hole (Appendix A) indicates that approximately 3 feet of clay and silt separate this deposit from underlying bedrock. A brief pump test performed on this hole resulted in 1.06 feet of head loss after 3.5 hours of pumping at 21 gpm (total available drawdown was 153.8 feet). Although the pumping rate and duration were inadequate for proper aquifer evaluation, the indication is that a high-capacity well might be developed in this formation. A chemical quality analysis of water from this test hole showed that it was of excellent quality for municipal use (see table on Figure 3.7).

(v) Summary

The 1979 municipal water-supply system of Elora consisted of two bedrock wells, with a combined rated capacity of 0.668 mgd. This supply can likely meet the average daily demand projected for the year 2001. To meet the average daily demand in 2031, based on a medium population projection, an additional 0.416 mgd may have to be developed. Although bedrock in this area is generally considered to

be a good aquifer, bedrock yields are highly variable. The buried bedrock valley to the south of Elora is recommended for future test drilling for potential municipal supplies.

4.3.3 Fergus

(i) Location and Physical Setting

Fergus is located at the junction of the Grand River and Highway No. 6, approximately 22 miles northeast of Kitchener (Figure 4.7).

Overburden in the area consists mainly of clay till interbedded with sand and gravel deposits at depth. Overburden thickness varies from less than 10 to about 100 feet over most of the area, and reaches a maximum of 259 feet in a narrow buried bedrock valley that runs generally north-south through the town. The sand and gravel deposits in this valley are up to 110 feet thick. The underlying bedrock in the area is dolomite of the Guelph Formation.

(ii) Existing Supplies

The 1979 municipal water-supply system of Fergus consisted of five bedrock wells with a combined rated capacity of 1.764 mgd. Well 836, rated at 0.360 mgd, is used only for emergency purposes because of its high sulphate and iron concentrations. The water from the municipal wells is generally very hard (up to 760 mg/L), exceeds 0.3 mg/L iron and 500 mg/L total dissolved solids, and has in the past had sulphate concentrations up to 600 mg/L. The 1977 average and maximum daily demands on this system were 0.615 and 0.900 mgd, respectively.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	8505	11133	13187
average daily demands (mgd)	0.876	1.147	1.359
maximum daily demands (mgd)	1.283	1.679	1.989
2031 population projections	11577	22810	32009
average daily demands (mgd)	1.193	2.351	3.299
maximum daily demands (mgd)	1.746	3.440	4.829

Note: The population projections for other years for Fergus are shown in Appendix B.

The 1979 supply of 1.764 mgd will likely be sufficient to meet the average daily demand projected for the year 2001. To meet the average daily demand based on medium population projection in 2031, an additional water supply of 0.587 mgd will be required.

(iv) Potential Future Supplies

A 1976 ground-water survey for Fergus (Andrijiw, 1976) recommended that additional supplies be sought from bedrock. Although many domestic wells in the area are developed in overburden, the sand and gravel deposits are generally too thin and too small in extent to yield sufficient quantities of water for municipal supplies. A possible exception to this may be in the buried bedrock valley that

runs through the northwestern section of Fergus. Saturated sands and gravels that overlie, or are located close to the bedrock surface in this valley, may yield sufficient quantities of water for municipal supplies. Future test drilling for municipal supplies is recommended in this buried valley.

The 'Ground-Water Yields From Bedrock' map (Figure 3.4) indicates the potential yield from bedrock in this area to be in excess of 200 gpm. However, the 1976 survey determined that differences in bedrock yields can be large even between neighbouring wells. This is attributed to large variations in the extent and depth of bedrock fracturing. Water of acceptable quality for municipal purposes may be expected from wells drilled to a depth of approximately 225 feet. Generally, deeper wells may produce higher yields but are likely to contain poorer quality of water.

A second area recommended for test drilling is located to the east of Fergus (Figure 4.7). The overburden in this area consists of about 50 feet of sandy to stoney clay till overlying grey limestone. Existing wells in the area report specific capacities of 0.7 gpm/ft to 2.6 gpm/ft, with available drawdowns ranging from about 80 to 140 feet.

(v) Summary

The 1979 municipal water-supply system of Fergus consisted of five bedrock wells with a combined rated capacity of 1.764 mgd. This supply can likely meet the average daily demand projected for the year 2001, but an additional supply of about 0.537 mgd will be needed to meet the projected average daily demand, based on a medium population projection, in the year 2031. The water in Fergus municipal wells is very hard, and exceeds drinking water criteria for iron, sulphate and total dissolved solids. Generally, yields from bedrock tend to increase with depth while chemical water quality may decrease. Additional municipal supplies may be developed in the basal sands and gravels in the buried bedrock valley, or in the bedrock in the valley. Another area recommended for test drilling in bedrock has been identified where the optimum well depth for suitable water quality appears to be approximately 225 feet.

4.3.4 Guelph

(i) Location and Physical Setting

The City of Guelph is located at the junction of the Eramosa and Speed rivers, approximately 12 miles northeast of Kitchener (Figure 4.2).

Overburden in the area is generally less than 100 feet thick, and consists of sandy silt till, overlain at the surface by outwash sands and gravels generally less than 20 feet thick. The most extensive outwash deposits on the surface are located in glacial spillways presently occupied by the Eramosa and Speed rivers, and to the southeast of their confluence. The underlying bedrock is dolomite of the Guelph Formation. This complex is recognized as one of the best high-capacity aquifers in Ontario, with yields in the Guelph area commonly up to 1000 gpm (Turner, 1978).

(ii) Existing Supplies

The 1979 existing municipal water-supply system of Guelph consisted of 20 bedrock wells, a shallow ground-water collector system near Arkell, and additional 7 bedrock wells that have been tested but have not been equipped with pumps (Figure 4.2).

The combined rated capacity of the 20 equipped wells is 15.40 mgd (Table 4.3). The collector system at Arkell yields approximately an additional 4.00 mgd throughout the year, while the system is supplemented from June to August by artificial ground-water recharge using water from the Eramosa River. This allows a seasonal recovery of ground water at the site of about 7.00 mgd. The combined rated capacity of the 7 additional (unequipped) wells is 6.41 mgd (Table 4.4). Consequently, the total capacity of the present system, including the 7 unequipped wells, is 25.81 mgd during fall, winter and spring, and 28.81 mgd during the summer months when the artificial ground-water recharge is in operation. The average and maximum daily demands on this system in 1977 were 9.50 and 13.10 mgd, respectively.

Table 4.3 Municipal Water-Supply System for Guelph, 1979

Production Well	MOE Well Number	Well Depth (ft)	Depth to Rock (ft)	Rated Capacity (mgd)
Eastview	888	296	58	0.43
Stone	2553	227	41	0.12
Park Street	858	156	26	1.46
Water	875	195	17	0.56
Edinburgh Road	873	228	24	0.36
Woodlawn	1333	314	19	0.32
Emma	856	225	31	0.37
University of Guelph	1439	211	54	0.50
Silvercreek	1377	220	17	0.09
Paisley	1380	263	38	0.72
Burkes	933	259	64	1.44
Queensdale	3649	244	27	1.15
Helmar	1132	261	38	0.62
Smallfield	1414	336	17	0.35
Dean	1429	188	39	0.43
Calico 4/76	6180	300	55	1.01
Carter	2699	65	21	1.44
Arkell 1/66	2769	67	66	0.72
Arkell 6/63	2808	145	34	1.44
Arkell 7/63	2766	142	74	1.87
Total rated capacity of production wells				15.40
Arkell collector system				4.00*
Total rated capacity of existing system				19.40*

* This value does not include an additional 3 mgd that is recharged and recovered in the summer only.

Table 4.4 Unequipped Wells for Future Municipal Supplies for Guelph

Test Well	MOE Well Number	Well Depth (ft)	Depth to Rock (ft)	Rated Capacity (mgd)
# 8	2809	138	38	1.01
Fleming	1127	221	47	0.58
Logan	1112	244	12	1.15
Hauser	954	210	37	0.22
Clythe 2/76	6103	210	12	1.15
McCurdy	2434	210	39	1.15
Downey	946	242	18	1.15
Total rated capacity				6.41

Ground water in the Guelph area is generally very hard, ranging from 200 to in excess of 400 mg/L. Iron is commonly in excess of 0.3 mg/L, and hydrogen sulphide is reported throughout the southern section of the city.

Information presented in an OWRC report on water resources in the Guelph area (Sobanski, 1968) indicates that there may be some difficulty in realizing the full rated capacity of the 1979 system. Assuming that the existing municipal wells draw ground water from within a radius of about 7 miles of the city, the total area affected by the existing system is about 154 square miles. Natural recharge in this area is estimated to be about 0.25 mgd per square mile (Figure 5.5). Thus, the total recharge to this area is about 38.5 mgd, of which only approximately one half (19.25 mgd) can be considered recoverable, the other half being needed to maintain baseflow to streams. The total rated capacity of the 1979 municipal system was 19.4 mgd, with an additional 6.41 mgd established but not yet utilized. It is possible that once the Guelph system exceeds 19.25 mgd in pumpage, it may be operating in excess of natural ground-water recharge. In this case, additional supplies may have to be developed outside the 7 mile radius discussed above.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	90,250	115,456	130,469
average daily demands (mgd)	12.18	15.58	17.61
maximum daily demands (mgd)	16.80	21.49	24.29
2031 population projections	121,643	209,133	273,669
average daily demands (mgd)	16.42	28.23	36.94
maximum daily demands (mgd)	22.64	38.93	50.94

Note: The population projections for other years for Guelph are shown in Appendix B.

The 1979 municipal water supply, including the 7 unequipped wells, appears to be sufficient to meet the average daily demand projected for 2001. To meet the average daily demands in the year 2031, based on a medium population projection, an additional 2.42 mgd will be required.

(iv) Potential Future Supplies

There are two potential sources for the development of additional water supplies in the Guelph area. These sources are the artificial recharge operation near Arkell and the additional development of ground water from the Guelph-Amabel Aquifer.

The Arkell artificial recharge operation currently withdraws 3 mgd from the Eramosa River during the summer months. This operation is licensed to extract up to 7 mgd from the Eramosa River, contingent on adequate river flows. A further expansion of this operation could increase the yield of the collector system to more than the 7.00 mgd it is designed for now. As part of this expansion, consideration might be given to operating the artificial recharge scheme throughout the year rather than only during summer months. However, any future expansion for seasonal or year-round operation of the recharge system will likely be governed by the amount of available water from the Eramosa River.

According to an OWRC survey of ground-water resources in 1968 (Sobanski, 1968), natural ground-water development may be possible almost anywhere in the Guelph area. This report states that topographically low areas such as spillways and outwash gravel areas offer better prospects of high yields than do topographically high till areas. As well, areas to the east of the Speed River appear generally better suited for municipal development than areas to the west because about twice as many wells with yields in the 1000 gpm range are located to the east of the river.

The "Ground-Water Yields from Bedrock" map (Figure 3.4) also identifies the area east of the Speed River as having a higher yield (from bedrock). This area is considered to have a good potential for well yields in excess of 200 gpm, compared to yields of up to 200 gpm west of the Speed River.

Accordingly, a potential test-drilling area for future municipal supplies has been identified southeast of Guelph (Figure 4.2). Existing wells in this area show 90 to 120 feet of clay, sand, gravel and till overlying limestone. Bedrock penetrations in these wells vary from 1 to 237 feet, with an average of about 60 feet. Specific capacities range from 2.0 gpm/ft to 150 gpm/ft, with an average value of 26.7. Available drawdown is generally about 100 feet.

This area is recommended for test drilling for the following reasons. First, well yields within this area may be expected to be in the order of 200 to 1000 gpm. Second, there are relatively few existing wells in this area, which may result in fewer well interference problems if the area is developed before the construction of municipal wells, and third, most of this area is within a mile of the present system.

It must be stressed that this area may not be the only potential location for future municipal water supply. Previous studies (Sobanski, 1968) have indicated that high-yield wells may be developed almost anywhere in the Guelph area. Also, this area is within the 7-mile radius discussed earlier, in which additional large-scale water takings could result in long-term aquifer mining. If this potential problem becomes a reality, test-drilling areas outside of the 7-mile radius will have to be identified.

(v) Summary

The 1979 municipal water system of Guelph consisted of 20 bedrock wells rated at 15.4 mgd, an additional 7 standby wells rated at 6.41 mgd, and a shallow ground-water collector system near Arkell that yields 4 mgd in winter, and 7 mgd in summer. This total supply is likely sufficient to meet the projected average daily demand in 2001. To meet the average daily demand in 2031, based on a medium population projection, an additional supply of about 2.42 mgd may be required.

There is a potential to expand the artificial recharge operation near Arkell, perhaps to double its present yield. As well, there is a potential for future ground-water development in an area southeast of Guelph where individual well yields of 200 to 1000 gpm may be possible.

It is important to be aware that increased municipal water takings close to Guelph may result in aquifer mining in the long term. This may lead to interference with private water supply systems and the eventual inadequacy of the Guelph water wells to meet future demands. From this point of view it is important to ensure that new ground-water development be carried out over a large enough area so that natural ground-water recharge is not exceeded by long-term pumping.

4.4 Communities Without Municipal Water-Supply Systems

In 1979 there were four communities within the Grand River basin that relied on private and communal wells for sources of water, and that are likely to establish municipal water-supply systems in the future. These communities are Burford, Drayton, Grand Valley and Salem. An evaluation of the present and future water supply/demand situation for each community follows.

4.4.1 Burford

(i) Location and Physical Setting

The community of Burford is located on Highway No. 53, approximately 24 miles south of Kitchener and 8 miles west of Brantford. Overburden thickness in the area varies from about 100 to 150 feet and consists of a sequence of interbedded silts, sands and clays, overlain by outwash sands and gravels of up to 60 feet thick. The underlying bedrock consists of shales and dolomites of the Salina Formation.

(ii) Existing Supplies

There was no municipal water-supply system in Burford in 1979. All private domestic wells in the area are in overburden, and most of these obtain water from the surficial outwash sands and gravels. Some wells obtain water from permeable deposits interbedded at various depths with silts and clays. There are no water quality problems reported for wells in the area.

(iii) Future Demand

In the absence of existing data on water consumption, average and maximum daily demands were assumed to be 100 and 160 gallons per day per capita, respectively. Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows:

	low	medium	high
2001 population projections	1025	1191	1348
average daily demands (mgd)	0.102	0.119	0.134
maximum daily demands (mgd)	0.164	0.190	0.215
2031 population projections	992	1383	1817
average daily demand (mgd)	0.099	0.138	0.181
maximum daily demand (mgd)	0.158	0.221	0.290

Note: Population projections for other years for Burford are shown in Appendix C.

(iv) Potential Future Supplies

An area recommended for future test drilling for municipal supplies is identified in Figure 4.6. This area is identified on the 'Ground-Water Yields from Overburden' map (Figure 3.5) as having a potential yield up to 200 gpm (0.288 mgd) per well. One such well would likely be sufficient to satisfy the average daily demand, based on a medium population projection, in the year 2031.

(v) Summary

The water supply of Burford in 1979 consisted of individual wells developed mainly in surficial outwash sands and gravels. Overburden in the area may yield up to 0.288 mgd per well and there appears to be sufficient potential in overburden in the area to meet the projected average daily demand in the year 2031.

4.4.2 Drayton

(i) Location and Physical Setting

The Village of Drayton is located in the northwestern portion of the Grand River basin along the Conestogo River, approximately 27 miles north of Kitchener.

Overburden thickness in this area varies from 150 to 200 feet and consists of clay and till interbedded with sands and gravels at various depths. As well, there are shallow surficial deposits of sand located on the Conestogo River floodplain. The underlying bedrock consists of 10 to 50 feet of blue shale over brown dolomite of the Salina Formation.

(ii) Existing Supplies

In 1966, the OWRC conducted a ground-water survey for Drayton to determine the potential for future municipal ground-water development (Morrison, 1966). The survey indicated that water supplies at the time were obtained from individual wells, most of which were constructed in the sandy alluvium of the Conestogo River floodplain (Figure 4.8). These shallow surficial deposits were found to yield sufficient quantities for individual domestic requirements, but showed no potential for municipal development. About 30 percent of the wells in the area reported fresh water from underlying dolomite at rates of about 10 to 20 gpm. The quality of water in the bedrock wells was found to be within acceptable limits for domestic use, with the exception of a few wells with high iron concentrations. The Drayton ground-water survey concluded that the dolomite bedrock underlying the shale constituted a favourable aquifer for municipal development.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are as follows. In the absence of existing water consumption data, average and maximum daily demands were assumed to be 100 and 160 gallons per day per capita, respectively.

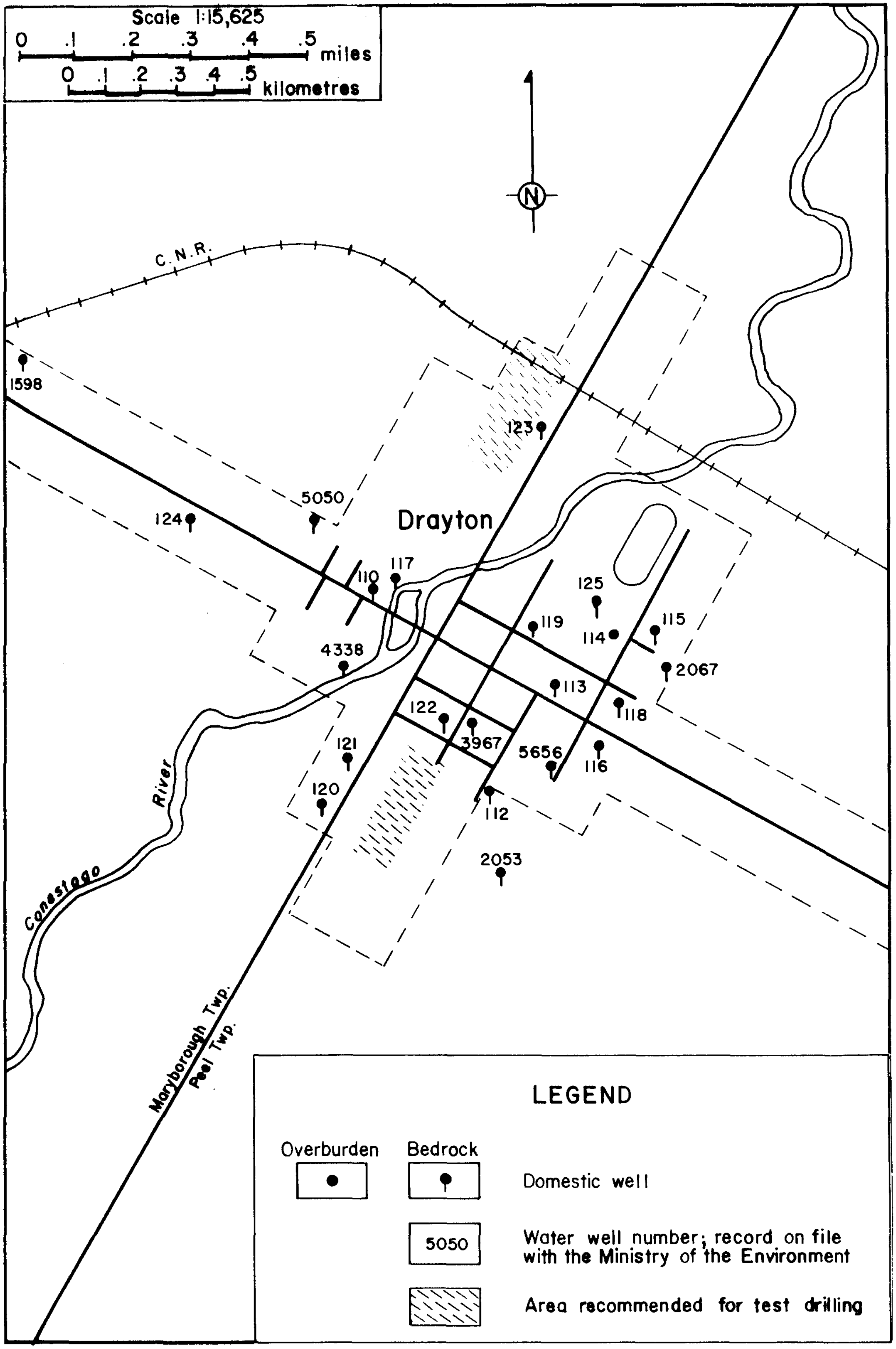


Figure 4.8. Locations of water wells and recommended test-drilling areas for Drayton.

	low	medium	high
2001 population projections	1042	1179	1507
average daily demands (mgd)	0.104	0.118	0.151
maximum daily demands (mgd)	0.167	0.189	0.241
2031 population projections	1405	1843	3162
average daily demands (mgd)	0.140	0.184	0.316
maximum daily demands (mgd)	0.225	0.295	0.506

Note: The population projections for other years for Drayton are shown in Appendix B.

(iv) Potential Future Supplies

In 1967, the OWRC drilled and developed a bedrock well to supply a future municipal system. Well 125 (Figure 4.8) was developed in dolomite at a depth of 218 feet and test pumped up to 250 gpm. The specific capacity of this well was calculated to be 13.3 gpm/ft, with an available drawdown of 200 feet. If only 50 feet of drawdown was used, well 125 could supply 0.957 mgd of water, which is more than five times the projected average daily demand in 2031, based on a medium population projection.

In the event that well 125 might be unable to meet the future demand, there are two areas that are recommended for future test drilling. The 1966 OWRC report concluded that most of Drayton has good potential in bedrock for municipal ground-water development and the two recommended areas represent optimum accessibility (in 1979) for municipal drilling and well development.

The northern area has approximately 240 feet of overburden and 40 feet of shale overlying brown dolomite. Water yield in this area is expected to be similar to that reported for near-by well 123 which has a specific capacity of about 6 gpm/ft and the total available drawdown is approximately 250 feet. A municipal well using less than one-fifth of the available drawdown could supply 0.432 mgd of water.

The southern area has approximately 160 feet of overburden and about 20 feet of shale over brown dolomite. The specific capacities and available drawdowns of near-by wells indicate that this area could yield in excess of 0.100 mgd per well.

(v) Summary

Well 125, drilled and developed by the OWRC in 1967, could satisfy the projected average daily demand in the year 2031. In case additional supplies are needed, two areas within the municipal boundary have been identified for future ground-water development.

4.4.3 Grand Valley

(i) Location and Physical Setting

The Village of Grand Valley is located in the northern portion of the Grand River basin, along the Grand River, approximately 37 miles north of Kitchener.

Overburden in the area consists mainly of till, with some localized sand and gravel deposits along the Grand River. The overburden thickness varies from 20 to 100 feet, but is generally less than 50 feet thick over most of the area. The underlying bedrock is dolomite of the Guelph Formation.

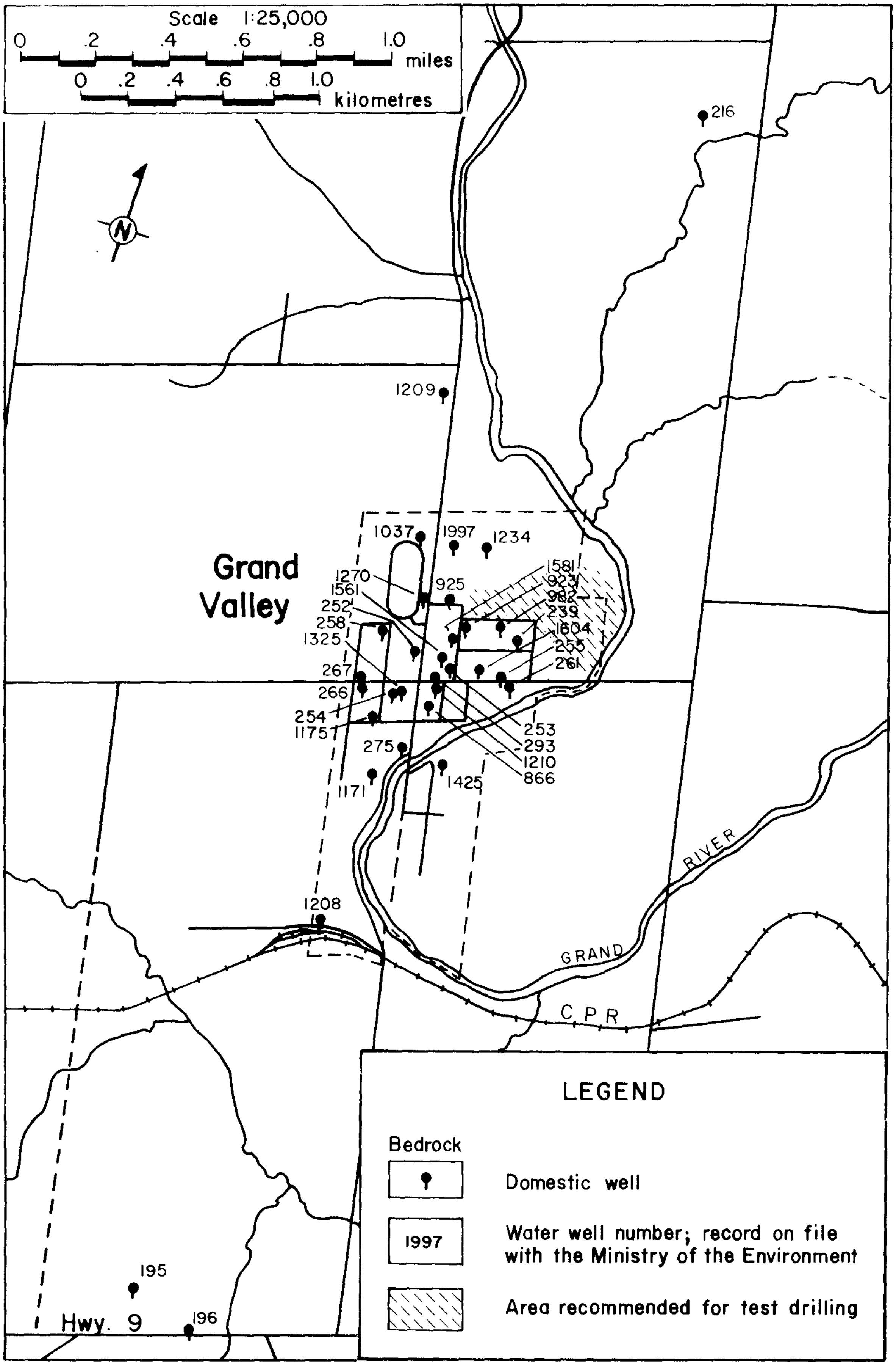


Figure 4.9. Locations of water wells and recommended test-drilling areas for Grand Valley.

(ii) Existing Supplies

Water supplies in Grand Valley in 1979 consisted of individual private wells, with the exception of a communal sub-division system in the northern end of the village, which is supplied by well 1997 (Figure 4.9). All but eight wells in the area are in bedrock, with an average penetration into rock of about 150 feet. All the wells in the area have specific capacities greater than 1.0 gpm/ft and there are no known water quality problems in the area.

(iii) Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections are shown below. In the absence of water consumption data, average and maximum daily demands were assumed to be 100 and 160 gallons per day per capita, respectively.

	low	medium	high
2001 population projections	1532	1739	2132
average daily demands (mgd)	0.153	0.174	0.213
maximum daily demands (mgd)	0.245	0.278	0.341
2031 population projections	2383	3150	4931
average daily demands (mgd)	0.238	0.315	0.493
maximum daily demands (mgd)	0.381	0.504	0.789

Note: The population projections for other years for Grand Valley are shown in Appendix B.

(iv) Potential Future Supplies

The 'Ground-Water Yields from Bedrock' map (Figure 3.4) identifies bedrock in the area to have a potential yield of up to 200 gpm (0.288 mgd) per well. The range of specific capacities of existing wells indicates that one municipal well could meet the projected average daily demand, based on a medium population, in the year 2031. For example, well 982 has a specific capacity of 7.5 gpm/ft and an available drawdown of about 150 feet. This well could satisfy the projected demands in 2031 using less than one-quarter of the total available drawdown. However, bedrock well yields can differ significantly even over short distances, and site-specific test drilling and test pumping must be carried out to determine individual well capacities and long-term yields. An area recommended for test drilling is shown in Figure 4.9. The overburden in this area is from about 25 to 90 feet thick, and the average bedrock penetration of near-by wells is about 125 feet.

(v) Summary

There was no municipal water-system for Grand Valley in 1979. Bedrock in the area appears to be a good aquifer, with probable yields of up to 200 gpm (0.288 mgd) per well. However, specific capacities of some existing wells indicate that yields in excess of 200 gpm are possible in some areas. Well development in the area recommended for test drilling should yield sufficient water to meet the average daily demand, based on a medium population projection, in the year 2031.

4.4.4 Salem

(i) Location and Physical Setting

Salem is located along Irvine Creek, approximately 20 miles northeast of Kitchener and less than one mile north of the Elora town boundary (Figure 4.2).

The physiography and surficial geology at Salem is similar to that described for Fergus and Elora. The overburden consists mainly of clay and till and varies in thickness from 0 to 110 feet. There are some local surficial sands and gravels up to 46 feet thick to the east and west of Salem. The underlying bedrock consists of dolomites of the Guelph Formation.

(ii) Existing Supplies

In 1979 there was no municipal water-supply system for Salem. Almost all of the private wells in the area end in bedrock, with penetrations of 20 to 225 feet. Although the specific capacities of the bedrock wells vary from 0.6 to 15.0 gpm/ft, almost 70 percent of the wells record specific capacities of less than 1.0 gpm/ft. The quality of ground water in the Salem area is considered good, although generally very hard (hardness values from 208 to 464 mg/L).

(iii) Future Demand

In the absence of existing data on water consumption, average and maximum daily demands were estimated to be 100 and 160 gallons per day per capita, respectively. Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are presented as follows:

	low	medium	high
2001 population projections	841	953	1,219
average daily demands (mgd)	0.084	0.095	0.121
maximum daily demands (mgd)	0.135	0.152	0.195
2031 population projections	978	1,284	2,208
average daily demands (mgd)	0.097	0.128	0.219
maximum daily demands (mgd)	0.156	0.205	0.353

Note: Population projections for other years for Salem are shown in Appendix C.

(iv) Potential Future Supplies

In response to a projected need for a municipal water-supply system in Salem, the MOE undertook a test-drilling program in the Salem area in 1975 (Ian Wilson and Associates, 1975). Test wells 6085 and 6087 were drilled and test pumped to determine their capability for yielding sufficient quantities of water for municipal requirements. The results of the tests indicated that there was mutual interference when both wells were pumped simultaneously, and that a combined safe yield for the two wells was 320 gpm (0.461 mgd).

This proven supply of 0.461 mgd would be more than sufficient to satisfy the 2031 average daily demand based on a medium population projection. Should additional supplies become necessary, test well 6084, which was drilled but not test pumped in 1975, is expected to yield moderate to large quantities of good-quality water.

(v) Summary

There was no municipal water-supply system in Salem in 1979. Test wells 6085 and 6087 were drilled and test pumped in 1975 as potential municipal supply wells. Their combined rated capacity of 0.461 mgd will likely exceed the average daily demand, based on a medium population projection, in the year 2031. A third test well (6084) was not tested but was considered likely to yield moderate to large quantities of water.

4.5 Water-Supply Options for Kitchener-Waterloo

4.5.1 Location and Physical Setting

Kitchener and Waterloo are adjoining cities located on the Grand River approximately in the centre of the Grand River basin. With a combined 1976 population of 181,773, this is the largest urban centre in the basin. The overburden thickness in this area varies

from about 100 to 350 feet and consists of complex sequences of outwash sands and gravels, glacial tills, and lacustrine silts and clays. Generally, the surficial deposits around Kitchener-Waterloo consist of sands and gravels, and generally clayey tills. The surficial sands and gravels vary in thickness from 10 to about 60 feet and are interbedded in some cases with tills. At depth, sands and gravels are interbedded with tills, and with silts and clays of various thicknesses. The underlying bedrock in the area consists of shale and dolomite of the Salina Formation, which is overlain by dolomite of the Guelph Formation to the east of Kitchener. The geologic boundary between these formations runs approximately north and south, separating Kitchener and Waterloo (Figure 2.1).

4.5.2 Existing Supplies

The existing municipal water-supply system for Kitchener-Waterloo consists of 29 wells in Kitchener, with a combined capacity of 22.242 mgd, and 12 overburden wells in Waterloo, with a combined capacity of 8.774 mgd (tables 4.5 and 4.6). Kitchener well K15 is the only bedrock well in the system and is not used except as an emergency standby supply because of high sulphate concentrations. The two Lancaster wells (K41 and K42A) also yield poor quality of water but not to the same extent as K15. The two Lancaster wells are pumped directly into the Kitchener-Waterloo system and are generally used during peak demand periods only.

In addition to natural ground-water supplies, Kitchener has two induced infiltration wells (K70, K71) along the Grand River south of Breslau (Figure 4.2 - the Forwell site). These induced infiltration wells are rated at about 0.816 mgd each, which increases the total Kitchener-Waterloo water supply to 32.648 mgd. The 1977 average and maximum daily demands on this system were 18.177 and 29.084 mgd, respectively.

Table 4.5 Municipal Water-Supply System for Kitchener, 1979

Well Number	Location	Rated Capacity (mgd)
K1	Greenbrook	0.720
K2	Greenbrook	0.576
K3	Greenbrook	0.144
K4	Greenbrook	1.008
K5	Greenbrook	0.636
K6	Greenbrook	0.244
K10	Strange Street	0.216
K11	Strange Street	0.864
K12	Strange Street	0.144
K13	Strange Street	0.504
K14	Strange Street	0.504
K15	Strange Street	Not Used
K16	Strange Street	0.288
K17	Strange Street	0.144
K18	Strange Street	0.806
K21	Mannheim	1.008
K22	Mannheim	1.008
K23	Mannheim	1.008
K24	Mannheim	0.972
K25	Mannheim	1.512
K26	Mannheim	2.016
K31	Parkway	1.008
K32	Parkway	1.008
K33	Parkway	1.008
K34	Strasburg	1.008
K41	Lancaster	0.144
K42A	Lancaster	0.720
K50	Wilmot	1.512
K51	Wilmot	1.512
Total (mgd)		22.242

Table 4.6 Municipal Water-Supply System for Waterloo, 1979

Well Number	Location	Rated Capacity (mgd)
W1B	William Street	0.252
W1A	William Street	0.547
W2	William Street	0.417
W2A	William Street	1.468
W3	William Street	0.835
W4	Laurel Lake	0.216
W5	Beaver Creek Road	1.008
W6	Erb Street	0.576
W7	Erb Street	1.656
W8	Erb Street	1.296
W9	Cedarbrae Crescent	0.086
W10	Hallman Road	0.417
Total (mgd)		8.774

4.5.3 Future Demand

Average and maximum daily demands in the years 2001 and 2031, based on low, medium and high population projections, are tabulated below.

	low	medium	high
2001 population projections	295,256	316,417	327,383
average daily demands (mgd)	29.52	31.64	32.74
maximum daily demands (mgd)	47.23	50.62	52.38
2031 population projections	324,852	451,577	519,399
average daily demands (mgd)	32.48	45.15	51.94
maximum daily demands (mgd)	51.97	72.26	83.10

Note: The population projections for other years for Kitchener and Waterloo are shown in Appendix B.

4.5.4 Potential Future Supplies

The 1979 municipal water supply of 32.648 mgd appears to be sufficient to meet the average daily requirements, based on a medium population projection, in the year 2001. An additional supply of 12.509 mgd will be required to meet the medium projection of average daily demand in 2031.

It appears doubtful that additional supplies could be developed from the well fields presently being used. Figures 4.10 to 4.16 indicate that water levels in these well fields have been dropping, in the case of the Parkway well field (Figure 4.10), by as much as 50 feet. Although a drop in water level in response to increased

pumpage is not unusual, the water levels in the Parkway and Strange Street well fields (figures 4.10 and 4.14) have dropped significantly even with relatively constant pumpages. As well, only the Mannheim and Erb Street well fields (figures 4.12, 4.13, 4.16) are pumped (1977 data) at the rated capacities of the respective formations. The remaining well fields are being pumped (1977 data) at one-third to two-thirds of their rated capacities. It is not known how much additional water-level lowering will result from pumpage increases that may be necessary to realize the full 32.648 mgd that the well fields are rated at. Although it is possible that the full rated capacity can be realized without serious problems (i.e. long-term aquifer dewatering), it is likely that these well fields can not sustain additional municipal-scale development.

In response to this situation, the Regional Municipality of Waterloo has undertaken studies to investigate water-supply options in the middle Grand area, particularily for Kitchener-Waterloo. One such option is the construction of relatively shallow wells near the Grand River in permeable deposits that are connected to the river. Subsequent pumping will induce Grand River water into these wells, as well as drawing natural ground water from the permeable deposits. As noted earlier, two such induced infiltration wells (K70, K71) have been constructed south of Breslau (at the Forwell site) and were in operation in 1979. Four smaller diameter wells have been constructed directly across the river from these two wells in an area known as the Pompeii site. The Pompeii wells are rated at a total yield of about 0.816 mgd but will be operated only during the peak period in the spring and summer months of each year. These wells are scheduled for operation in 1980.

An additional three induced infiltration wells have been constructed along the Grand River south of the Forwell site in an area known as the Woolner site (Figure 4.2). These wells are rated at a total capacity of 2.448 mgd and are scheduled for production in 1981. There is one additional induced infiltration well proposed along the

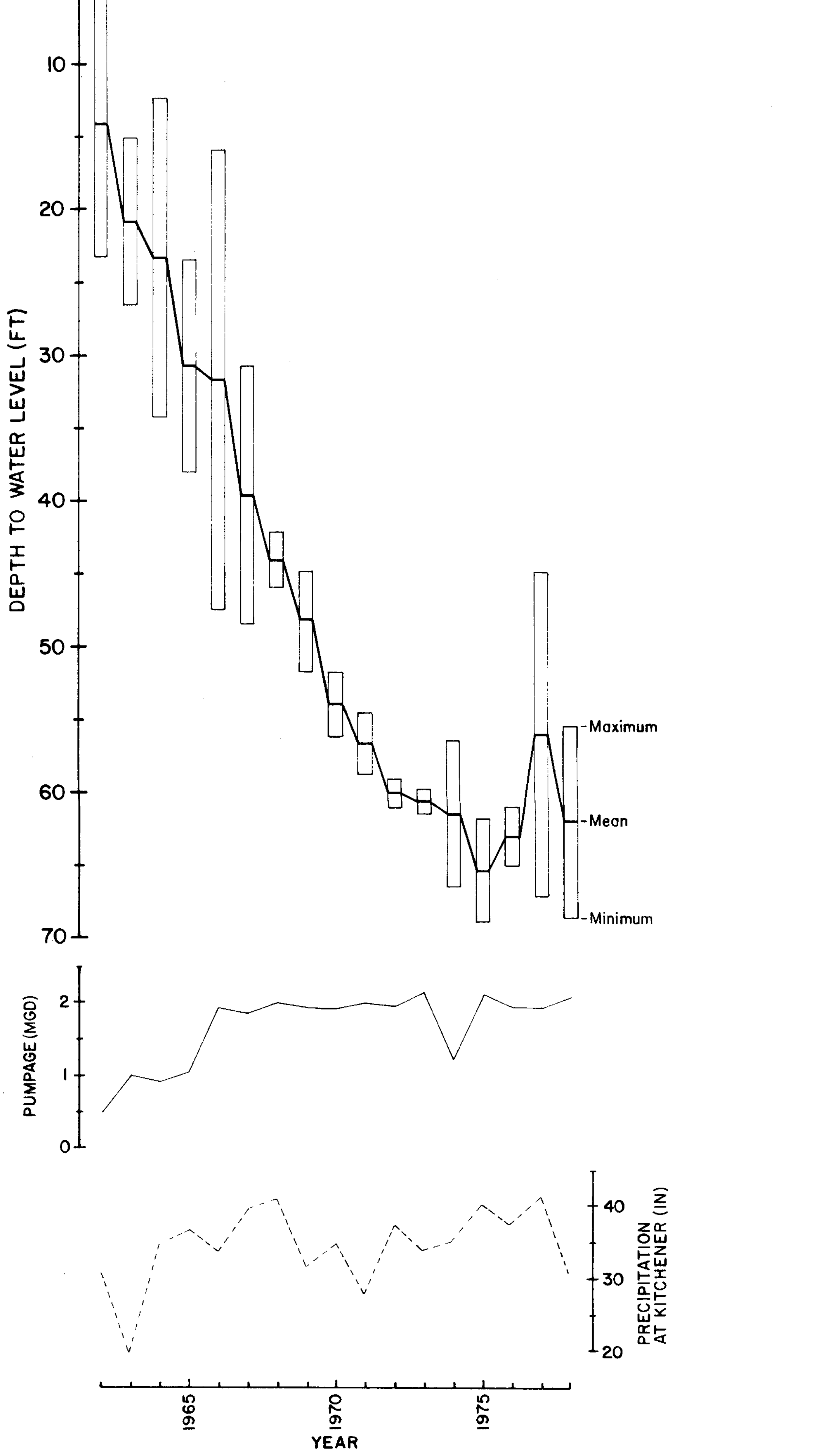


Figure 4.10. Water levels at Observation Well 82 and pumpage at Parkway well field (wells K31-K33 inclusive).

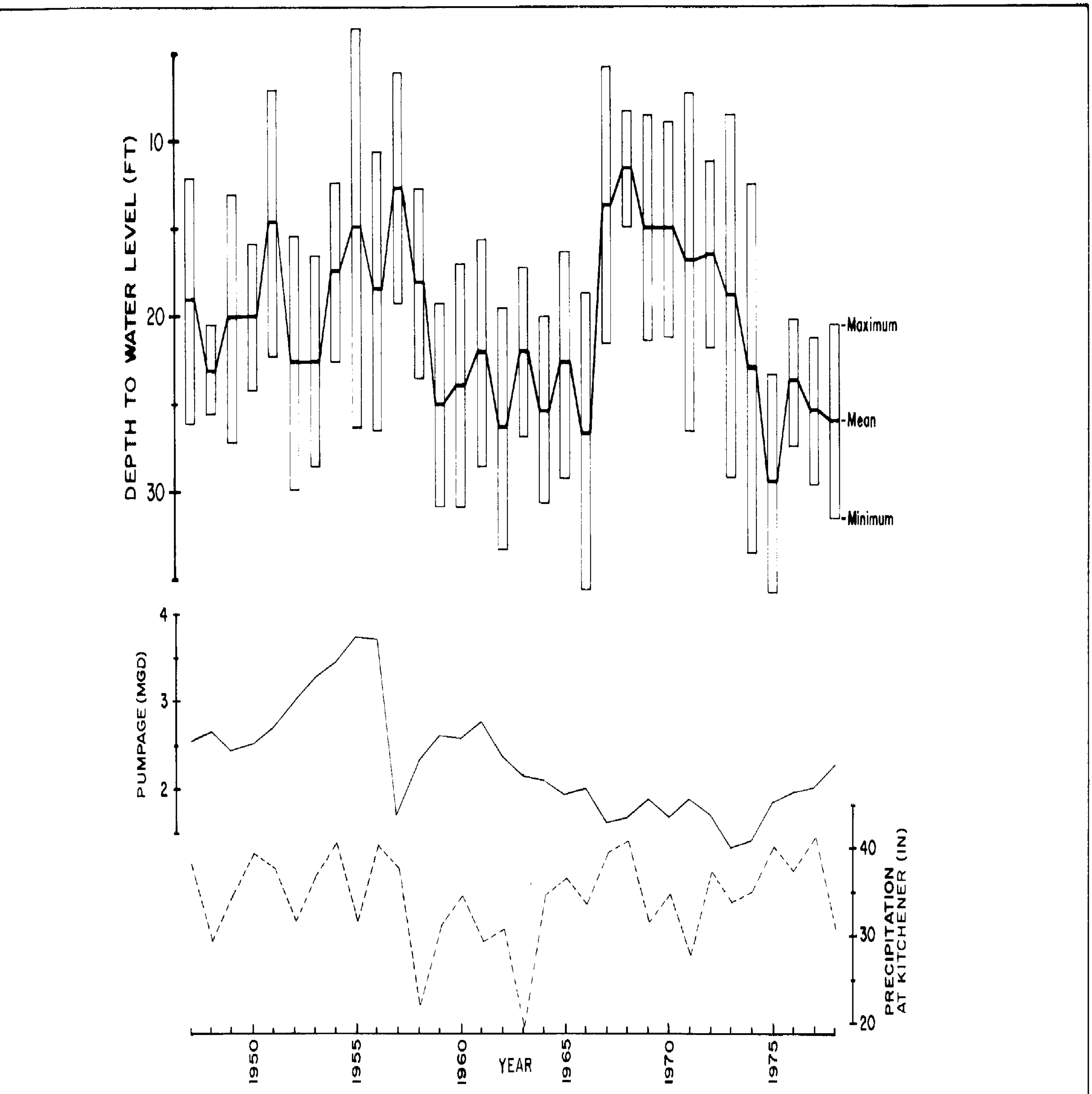
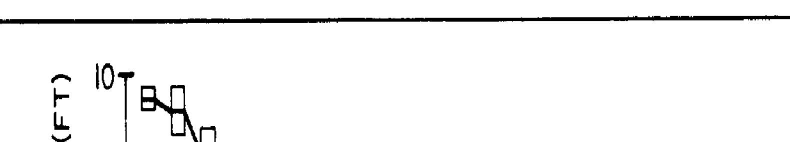


Figure 4.11. Water levels at Observation Well 35 and pumpage at Greenbrook well field (wells K1-K6 inclusive).



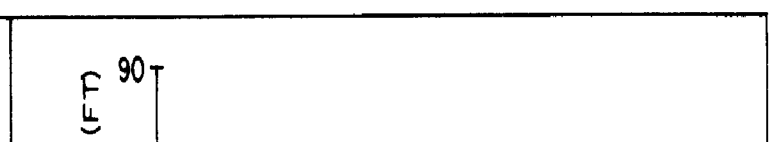


Figure 4.11. Water levels at Observation Well 35 and pumpage at Greenbrook well field (wells K1-K6 inclusive).

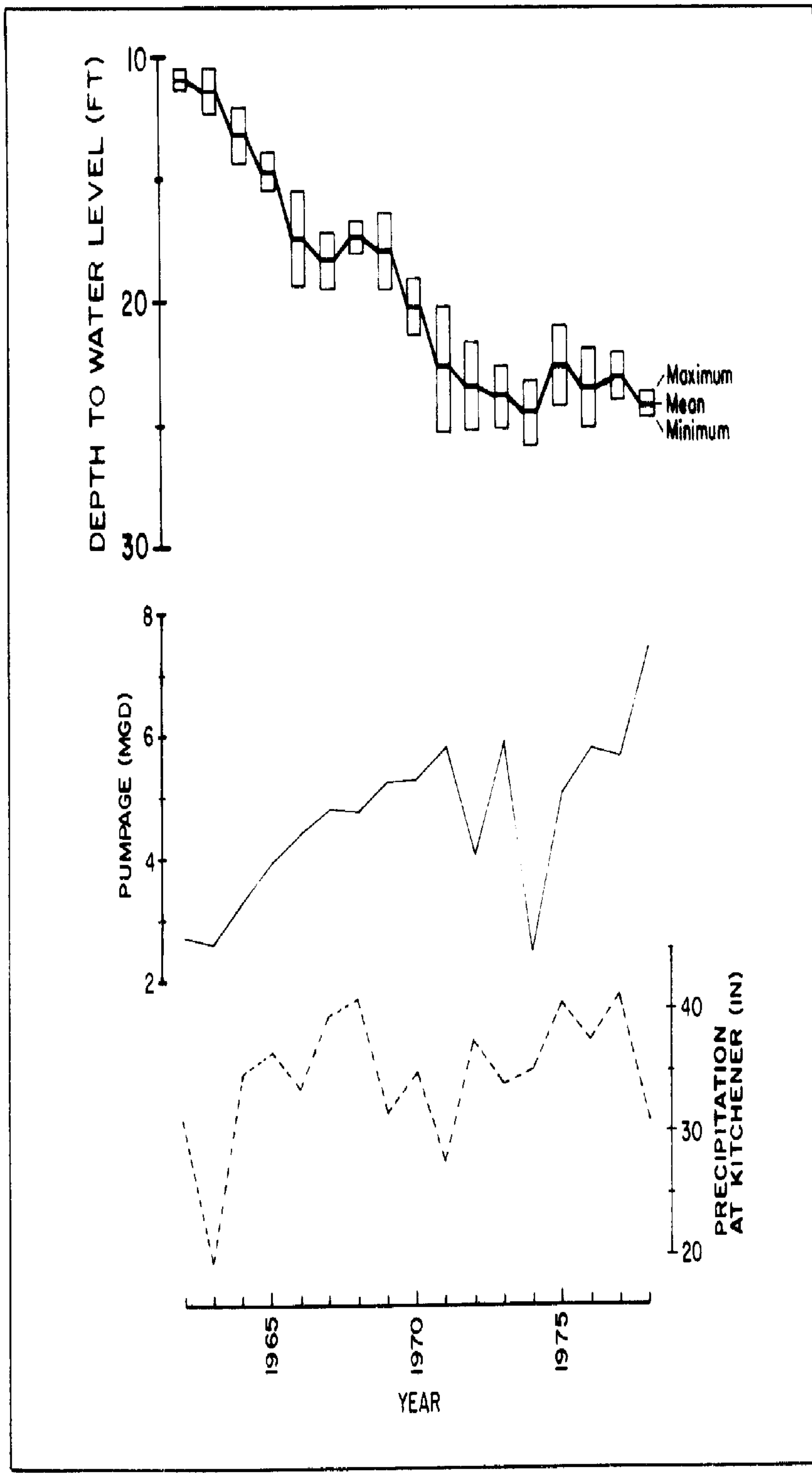


Figure 4.12. Water levels at Observation Well 117 and pumpage at Mannheim well field (wells K21-K26 inclusive).

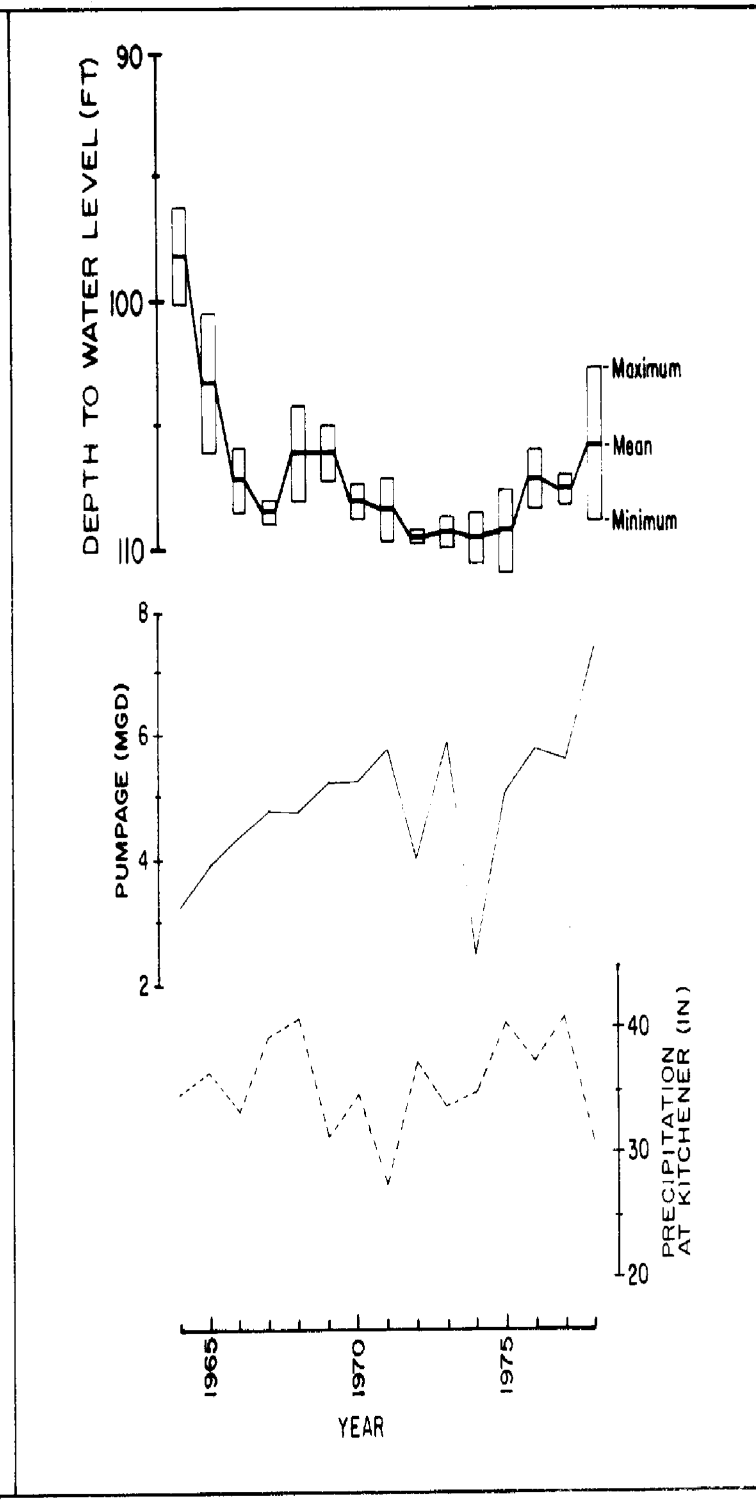


Figure 4.13. Water levels at Observation Well 116 and pumpage at Mannheim well field (wells K21-K26 inclusive).

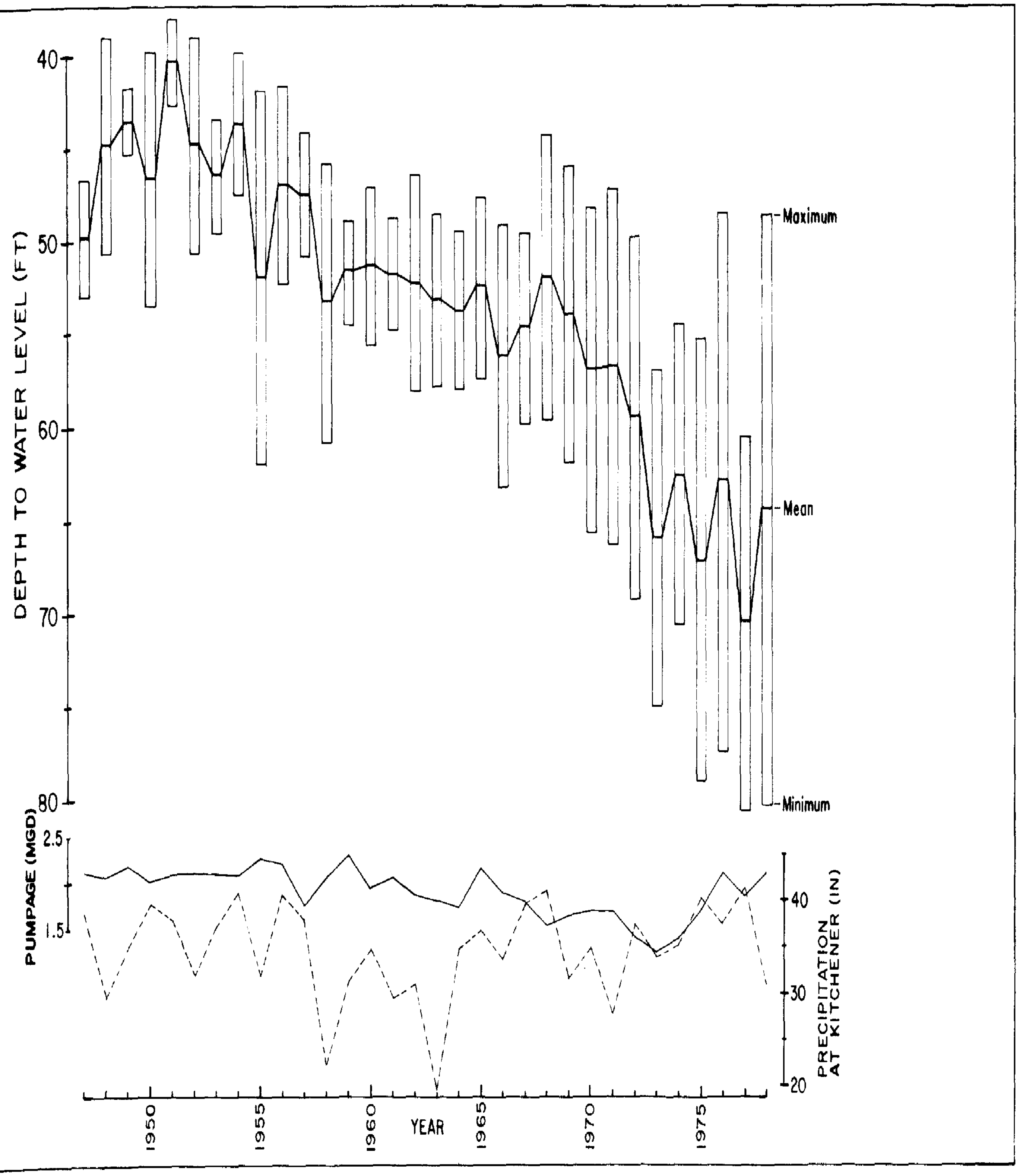


Figure 4.14. Water levels at Observation Well 59 and pumpage at Strange St. well field (wells K10-K18 inclusive).

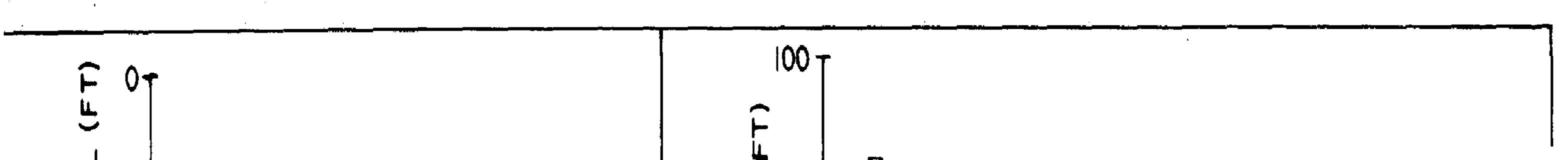


Figure 4.14. Water levels at Observation Well 59 and pumpage at Strange St. well field (wells K10-K18 inclusive).

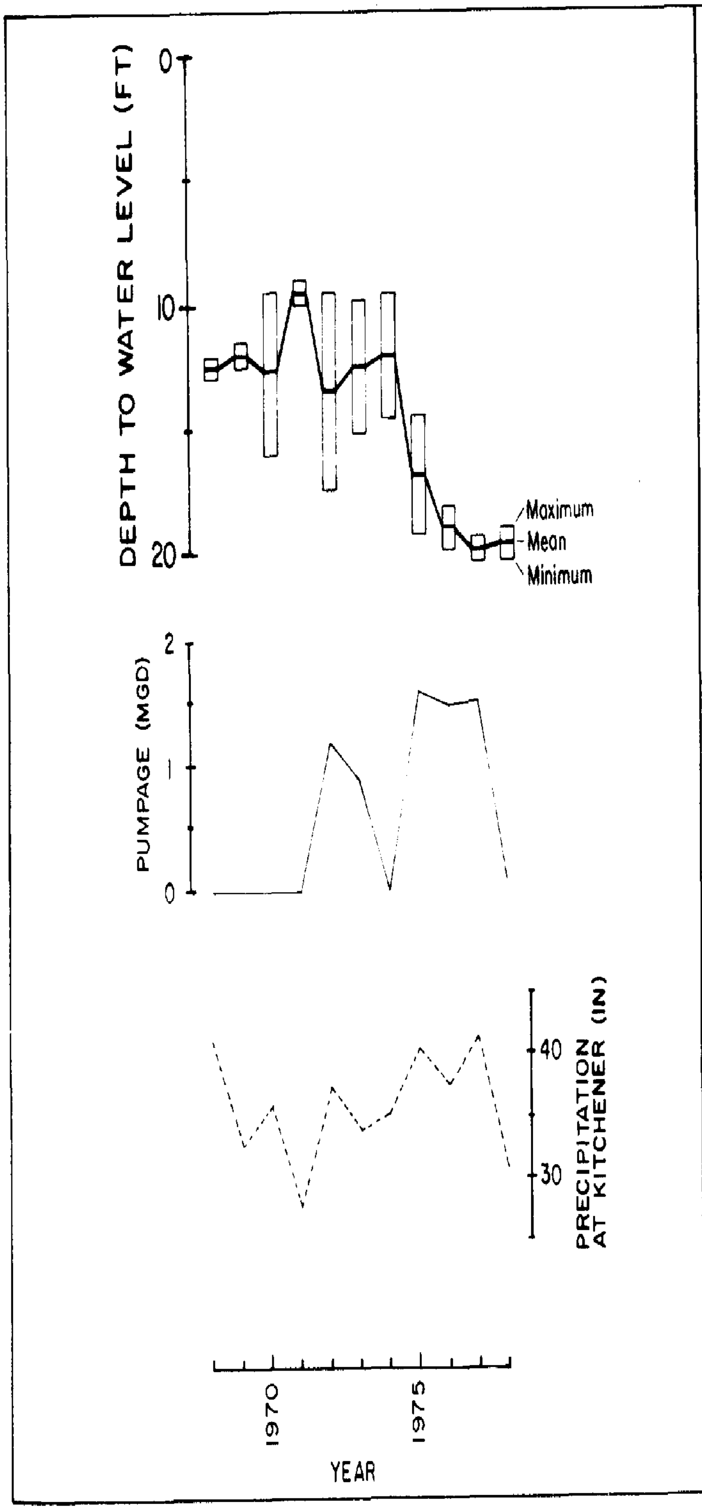


Figure 4.15. Water levels at Observation Well 2188 and pumpage at Wilmot well field (wells K50 and K51).

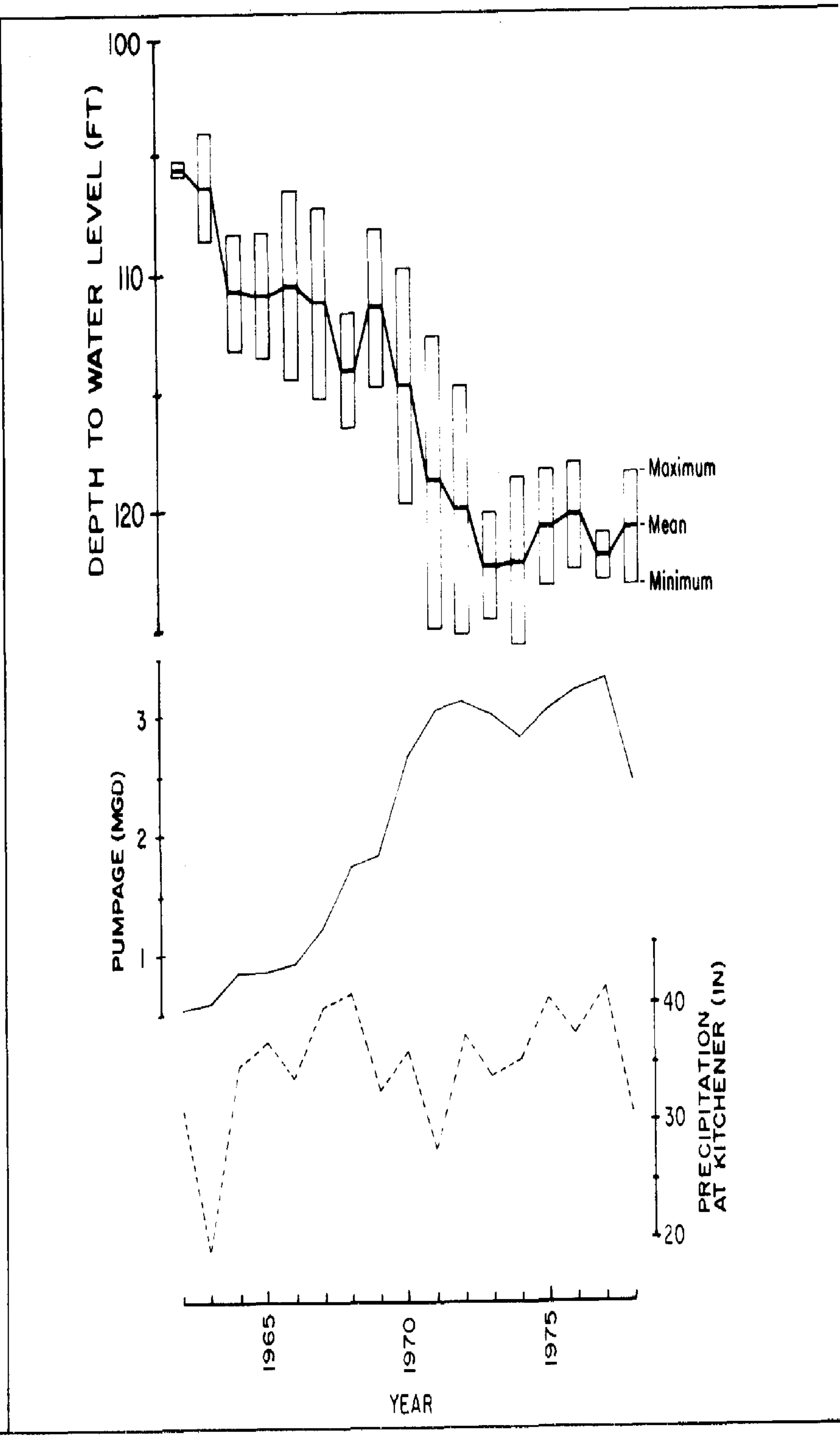


Figure 4.16. Water levels at Observation Well 2340 and pumpage at Erb St. well field 'wells W6-W8 inclusive).

Grand River just north of Breslau (the Breslau site), and with an estimated capacity of 1.008 mgd, is tentatively scheduled for operation in 1983. The total number of induced infiltration wells constructed and/or proposed is 10.

In addition to the induced infiltration projects, the Regional Municipality of Waterloo is also engaged in a pilot study of artificial ground-water recharge near Mannheim. Previous investigations (Hydrology Consultants, 1974) recommended two sites for investigation of artificial ground-water recharge; near Roseville and near Mannheim. Since an artificial recharge facility near Roseville would depend on the proposed Ayr reservoir as a raw water source, the Municipality is first considering a facility at Mannheim. A pilot project was undertaken in December of 1979 at the site and this will extend into 1980. It is estimated at this early stage that about 20 mgd could be recharged, using water from the Grand River, and recovered at Mannheim. It is expected that initially this recharge facility would be operated for at least 3 months of the year in the spring and early summer when river flows are high. At some later stage, a year-round operation of the scheme is likely, provided adequate water is available from the Grand River. A project evaluation, with design criteria and individual costs, is included in Appendix F.

Another artificial recharge option is considered feasible in an area southeast of Roseville. This development could be designed to yield at least about 20 mgd and is dependant on raw water from a reservoir proposed on the Nith River northwest of Ayr. A project evaluation, with design criteria and individual costs, is included in Appendix G.

The development of new well fields is another water-supply option to be considered for Kitchener-Waterloo. An examination of hydrogeologic data and water well logs south of Kitchener has indicated that a potential high-yield aquifer is located north of Roseville (Figure H.1). A water-bearing sand and gravel formation, 10 to 30 feet thick, at a depth of about 110 feet, is being tapped by wells in the area. This formation was confirmed in two test holes drilled in July and August 1979. The preliminary analysis of data indicates that the aquifer extends over about 10 square miles and has a potential to yield about 3 mgd. A test-drilling program with intensive test pumping will be necessary to determine the actual potential of this area as a municipal well field. A project evaluation for this option, with design criteria and individual costs, is included in Appendix H.

The following is a summary of the water-supply options for Kitchener-Waterloo, as discussed above:

Present capacity of Kitchener-Waterloo wells - 31.016

Project	Implementation	Capacity
A. Induced Infiltration:		
Forwell (2 wells)	operational (1979)	1.632
Pompeii (4 wells)	1980	0.816 (peak supply)
Woolner (3 wells)	1981	2.448
Breslau (1 well)	1983	1.008
B. Artificial Recharge:		
Mannheim	-	20 (peak supply)
Roseville	-	20 (peak supply)
C. Natural Ground-Water Development:		
Roseville	-	2.88

There are two other potential sources of water for Kitchener-Waterloo which have not been included among the water-supply options discussed so far because they are contingent upon the construction of the proposed West Montrose reservoir, and would require a pipeline connection between Kitchener-Waterloo and Elmira. The first of these options is an induced infiltration development around Inverhaugh, which is along the Grand River, approximately two and one-half miles south of Elora. A preliminary engineering report on the proposed reservoir (Tomlinson and Associates, 1967) has determined that the water elevation within the proposed reservoir will be between 1140 and 1152 feet. An examination of existing water well logs, coupled with a cursory test-drilling operation in the area in 1979, has indicated that there is a permeable sand formation, about 20 feet in thickness, that is at a lower elevation than 1140 feet, and which may be connected to water in the reservoir. In the event that this does occur, wells developed in the sand formation adjacent to the reservoir might induce water from the reservoir. Water from this development could be transported to Elmira via a pipeline. At present there is a pipeline from Elmira to St. Jacobs, and an extension of the Kitchener-Waterloo system to St. Jacobs would make the Inverhaugh supply available to Kitchener-Waterloo. In the event that the West Montrose reservoir is built, the potential for induced infiltration in the Inverhaugh area should be investigated further.

The second possible water-supply option for Kitchener-Waterloo is an artificial ground-water recharge development about three miles north of Elmira and two and one-half miles northeast of Floradale. The site contains fine to coarse surficial sands that have a maximum thickness of about 105 feet. Existing well records and geologic data indicate that the sands have a saturation of about 40 feet and are underlain by about 85 feet of silt and till. A more thorough and extensive evaluation is recommended if the proposed West Montrose reservoir, which would be the source of raw water for this development, becomes a reality. As in the case of the Inverhaugh project, the recovered ground water could be piped to Elmira-St. Jacobs, and via a connecting pipeline, to Kitchener-Waterloo.

4.5.5 Summary

The 1979 municipal water-supply system for Kitchener-Waterloo consisted of 41 ground-water wells with a combined rated capacity of 31.016 mgd, and two operational induced infiltration wells rated at 1.632 mgd. There does not appear to be much potential for additional development of the existing municipal well fields. The Regional Municipality of Waterloo is committed to the construction of 8 additional induced infiltration wells. Four of these, rated at 0.816 mgd, will be used during the summer peak demand period, and four others, with a combined rated capacity of 3.456 mgd, will be used on a continuous yearly basis. Assuming that all these developments are realized, the average daily water supply capacity for Kitchener-Waterloo may be rated at 36.104 mgd.

The Regional Municipality has also undertaken a pilot study of artificial ground-water recharge near Mannheim. Preliminary results indicate that 20 mgd may be recharged at this site during the spring and early summer months. Additional water-supply options, including natural ground-water development near Roseville, artificial recharge near Roseville and Floradale, and induced infiltration near Inverhaugh are all future possibilities that could increase significantly the capacity of the existing water-supply system. However, considerble exploratory work and analysis would need to be done before the yields of any of the new systems could be established. Several of the projects depend on the construction of new reservoirs.

5. GROUND-WATER MANAGEMENT ISSUES

5.1 Introduction

Rural water supplies in the basin depend almost exclusively on ground water and most of the municipalities utilize ground water to satisfy their communal needs. Obtaining rural domestic needs is generally not a problem in most parts of the basin because the individual needs are usually small and the demand is distributed relatively evenly throughout most of the basin. Communal needs, on the other hand, often require large-scale, concentrated takings and adequate quantities of good quality water are not always available close to each community. The expanding needs of communities makes the search for new supplies an ongoing task at an increasing radius from each community, resulting at times in interference with established ground-water takings.

The issue of sufficient supplies for municipal needs in the basin is rendered more complex by the occurrence of poor natural water quality in certain areas, and by the depletion of local aquifers due to over pumping by high capacity wells. In yet other situations, local aquifers are rendered unfit for use because of contamination from certain land use practices normally associated with urban development. As urban activities increase throughout the basin, the chances of contamination of ground water from surface sources are increased and this possibility needs to be recognized and dealt with at initial stages of land use planning.

The search for the most economic water supplies for Kitchener-Waterloo is an ongoing activity as increased urbanization creates greater demands for water. The existing supplies within the boundaries of the cities have been developed to capacity and the radius of search for new supplies has expanded outside the municipal bounderies, resulting in some situations in interference with local domestic water supplies. The increased radius has also produced economic limits on the transportation of water to the cities, and consequently, current efforts in securing new water supplies have

been directed at utilizing close-by surface-water resources. In this context, the utilization of water from the Grand River for induced infiltration and artificial ground-water recharge systems are considered viable methods of obtaining additional water supplies, and will ultimately provide a greater flexibility for the planning and development of future supplies in the twin cities area.

5.2 Poor Natural Water Quality

Most occurrences of poor natural ground-water quality relate to supplies obtained from bedrock, or from overburden formations close to bedrock. The most common problem is undesirable concentrations of sulphates and often the presence of hydrogen sulphide gas. In addition, some ground waters are very hard, some have a salty taste, and yet others contain high concentrations of iron. However, all of the latter problems are less widespread and generally of less concern for rural or municipal uses than the occurrence of high sulphates.

Figure 5.1 indicates the locations of wells with poor water quality as reported by water-well drillers (in well logs) at the time of drilling of the wells. The reported water quality problems range from general terms such as "mineralized", to specific terms such as "salty", and by far the most often used term is "sulphurous". This is taken to indicate the presence of high concentrations of sulphates and possibly detectable amounts of hydrogen sulphide gas.

Almost all water quality problems shown on Figure 5.1 in the southern third of the basin relate to sulphurous waters found at various depths in bedrock. Most of the wells in the area are for domestic uses. Standards for municipal use dictate that where better quality water is available, waters with concentrations of sulphates greater than 250 mg/L should not be used for distribution. This limits the use of some ground waters for municipal development, and for this reason Kitchener-Waterloo has not searched for new supplies east of the twin cities where the occurrence of sulphurous waters in bedrock is common. In spite of this, Guelph to the east of Kitchener- Waterloo has successfully developed adequate amounts of good quality water in an area that can

also produce waters containing high sulphates. In fact, several of the Guelph municipal wells are capable of producing high sulphate waters under conditions of large pumping.

Treatment for sulphate removal at the municipal scale is generally not economical as indicated by the past situation at Plattsville. The newly developed municipal wells (in 1979) in the town contained approximately 580-840 mg/L of sulphates at the time of construction, and the reduction of these concentrations by reverse osmosis was planned initially. However, excessive costs to consumers for this treatment were subsequently considered prohibitive and the town will use the water without reduction of the sulphates.

Of somewhat secondary concern regarding water quality reported by drillers are mineralized and salty waters. Most of these waters are scattered throughout the basin, with a density of wells containing mineralized waters in the area southwest of Kitchener between New Hamburg and Paris. Mineralized waters are assumed to indicate high hardness and to contain other parameters such as iron and bicarbonate in objectionable quantities. In most cases, these waters can be made suitable for use by readily available domestic treatment systems.

A noticeable feature on the map is the apparent lack of water quality problems in the northern part of the basin. Although part of this may be due to the fact that generally fewer wells are developed in the northern area, it is probable that ground-water problems in either overburden or bedrock are not common.

5.3 Flowing Wells, Dry Wells

Flowing wells and wells reported to yield insufficient water for even domestic purposes are of secondary concern in most areas of the basin. Consequently, the inventory of their occurrence (Figure 5.2) is presented primarily for general information in order to briefly discuss some of the management-related issues with each.

Flowing wells in the basin are constructed in both overburden and bedrock and consequently there is a wide variety in the depths of these wells. Most of the flowing wells in the northern part of the basin end in bedrock and vary in depths up to about 350 feet; however, the majority of the wells are less than 250 feet deep. In the central part of the basin in the vicinity of Kitchener-Waterloo, Cambridge and Guelph, a greater proportion of wells are completed in overburden, but bedrock wells are still most common (Figure 5.2). Overburden well depths in this area vary up to about 180 feet, with the majority being less than 100 feet deep. Bedrock wells are only slightly deeper, varying up to about 280 feet in depth, but averaging only about 100 feet. There are only a few flowing wells in the southern part of the basin.

The management of flowing wells involves two basic issues. One relates to the conservation of ground water and the other to surface drainage of water from flowing wells. Every well that is allowed to flow freely, with part or all of the discharge water unused, represents a waste of ground water per se. The cumulative effect of many flowing wells in an area tapping the same aquifer is to reduce the hydrostatic pressure to the point where some or all of the wells may stop flowing. Once the flow has been stopped, some wells may need to be reconstructed or new ones drilled to accommodate a pump that may not have been previously necessary. By reducing or totally stopping the flow from wells, streamflow that benefitted from the surface discharge of ground water will in turn be affected, resulting in various degrees of interference with established streamflow uses. This can affect various in-stream uses such as fisheries and other recreational activities, and result in less water available for stream-water withdrawals, and possibly in changes to water quality.

The surface discharge of water from flowing wells can also create land damage through erosion, produce problems associated with surface drainage in low-relief areas, and in some instances create nuisance aesthetic conditions in drainage ditches. Land erosion problems are most prominant when a well first starts to flow, and the situation can become serious if the flow cannot be controlled or

diverted to prevent damage to buildings and neighbouring property. Flooding of low-relief areas downstream from the flow can limit the use of otherwise productive agricultural lands, and water in drainage ditches can be productive to the growth of algae and other nuisance aquatic growth.

To help prevent the uncontrolled flow from wells, Ontario Regulation 648/70 requires that "...the contractor ... install a device that is capable of controlling the discharge of water from within the well casing." However, since it is not mandatory that the flow of water be restricted to the useful part of the flow, i.e., allow only flow equivalent to the actual consumptive use, the Regulation is not always effective in conserving water, or preventing or resolving problems associated with flowing wells. This can be achieved only through the co-operation of owners of flowing wells concerned with water conservation and ground-water management.

The majority of wells in the basin reported by drillers to be "dry" at the time of construction are domestic wells. The wells are generally shallow, 40 - 50 feet deep, although dry wells close to 250 feet deep in overburden have been reported west of Kitchener-Waterloo. It appears that most of the shallow wells are not deep enough to intercept water-bearing formations capable of providing adequate yields. This is especially true if the wells have been drilled rather than bored.

Although there is generally a good occurrence of ground water from both overburden and bedrock formations in most areas of the basin, ground water in the clay plain area in the southern part of the basin can be difficult to obtain locally for even domestic purposes. The predominance of dry wells in bedrock north of Caledonia suggests that there are no water-bearing formations in the overburden in the area and that the bedrock is also not a reliable source of water for even small requirements such as domestic supplies. The area also contains poor quality (sulphurous) ground water.

5.4 The Susceptibility of Ground Water to Contamination

The dependence of rural and municipal water supplies on ground water makes the protection of the resources from contamination paramount. Unfortunately, in urban areas where ground water plays a significant role there are also the greatest number of potential sources of contamination. In rural areas where the population is less dense, ground-water contamination may affect fewer individuals but still be significant in terms of restricting future utilization of ground water.

There are many sources of pollution, and correspondingly a large number of physical factors that combine to make the issue of contamination very complex. Contamination sources, for example, can vary from diffuse sources such as agricultural fertilizers and other farm-related operations, to specific point sources such as accidental spills of liquid contaminants, leachates from landfill sites, or the infiltration of snowmelt runoff containing deicing chemicals. The permeability of the different soils and bedrock is a major factor that determines the rate and amount of infiltration of contaminants, and consequently the attenuation of contaminants once in the ground.

Appreciating the complexity and variability of specific cases of contamination of ground water, the intent of the present discussion is to define, in very broad terms, the areas having a high potential to contamination from surface sources. It is important to stress that the present inventory is general and is in no way meant to indicate the contamination potential of an area to specific land uses. This can be achieved only through site-specific studies that deal with individual contaminating substances and local hydrogeologic conditions in order to determine the likely potential of contaminating ground water.

Four general environments in the basin are apparent in describing the susceptibility of ground water to contamination from surface sources; three environments have generally a high susceptibility and one has a low susceptibility (Figure 5.3). In defining these areas, the permeability of surface soils was considered most important, assuming that permeable formations such as sands, gravels and dolomite/limestone bedrock at or near the surface would readily allow contaminants to enter the ground-water system. Clays, silts and till materials, on the other hand, have generally low permeability and therefore allow little infiltration of contaminants into the ground.

The distinction between areas containing bedrock covered by thin overburden or exposed at the surface (Area 1), and areas of surficial sands and gravels (Area 2), reflects primarily the difference in the attenuation capacities of the materials and the rates of movement of contaminants. In general, the rate of movement is higher and the degree of attenuation is smaller, per unit distance travelled, in dolomite/limestone than in sands and gravels. Consequently, coupled with the fact that the Guelph Formation dolomite is a productive aquifer, the prevention of contamination in shallow or exposed dolomite areas in the basin is most critical.

For the most part, the legend on Figure 5.3 is self-explanatory, with the following supplemental notes pertaining to Area 2:

1. the area east of Guelph contains the Arkell artificial recharge and municipal collector system;

2. the area north of West Montrose has the potential for induced infiltration wells in association with the proposed West Montrose reservoir;

3. the area south of South Woolwich has the potential for artificial recharge using water from the proposed West Montrose reservoir;

4. the area near Conestogo contains sites potentially suitable for induced infiltration of Grand River water; the same applies for the two small areas south of Kitchener along the Grand River;

5. the area near Breslau contains induced infiltration wells K70 and K71, and two other proposed sites for the development of induced infiltration by Kitchener-Waterloo;

6. the area near Mannheim will contain an artificial recharge site for Grand River water;

7. the area near Roseville is proposed for artificial recharge associated with the proposed Ayr reservoir;

8. the area north of Paris contains the municipal collector system for the town.

The areas having generally a low susceptibility to contamination contain surface materials of low permeability such as tills, silts and clays. These materials are usually thick enough in the basin to provide a protective cover for aquifers at depth. In the general area southeast of Brantford, there are no significant (high capacity), extensive aquifers under the surficial clays, and consequently the chances of contaminating significant supplies of ground water in the area are negligible.

The issue of ground-water contamination in all its facets is complex and involves rapidly evolving technology, making the scope of above discussions cursory at best. However, the classifications shown on the map should provide a sense of awareness of the problem that in the future may provide a constructive base for municipal land use planning. The map is only a first step in overall ground-water resources management planning in the basin. Subsequent steps related to ground-water contamination must, by necessity, involve site-specific evaluations of hydrogeology in relation to specific land use activities that may pose a potential hazard for ground-water contamination.

5.5 Other Ground-Water Environmental Issues

Increased concern for ground-water conservation is evident in many areas in the basin. Some concerns are the result of interference with private domestic supplies, while others are based on a more general awareness of the resource and the potential effects of some land uses on it. To illustrate the need for conservation, an inventory of past known cases of interference with ground-water supplies was made (Figure 5.4). Results of this inventory indicate that past interference cases have involved both ground-water quantity and quality (Table 5.1). Interference with ground-water quantity has most often been produced by takings from large-capacity wells, while interference with quality has predictably resulted from activities associated with urban-type developments.

A number of complaints of interference related to quarry or gravel pit dewaterings, and to dewaterings associated with various construction activities in the basin, have been documented. However, the majority of complaints have been associated with large-capacity municipal wells. Of the complaints related to municipal takings, those related to Kitchener-Waterloo wells have been most prevalent (Table 5.1), and have usually involved lowered water levels in domestic wells, and in several instances have resulted in drying up of surface streams. Similar complaints, but fewer in number, have also been associated with municipal wells for Guelph, Cambridge and some of the other smaller municipalities generally in the central part of the basin.

All valid complaints of water-level interference have been resolved through the Permit to Take Water program of the Ministry of the Environment and its related interference policies. The Permit program is based on Provincial legislation (s. 37, Ontario Water Resources Act) that makes it manditory for takings of surface and ground waters in excess of 10,000 gallons per day to be permitted. Takings for ordinary domestic and farm purposes, and for fire fighting, are exempt. A condition of granting a Permit is that if a taking interferes with existing water supplies,the permittee is

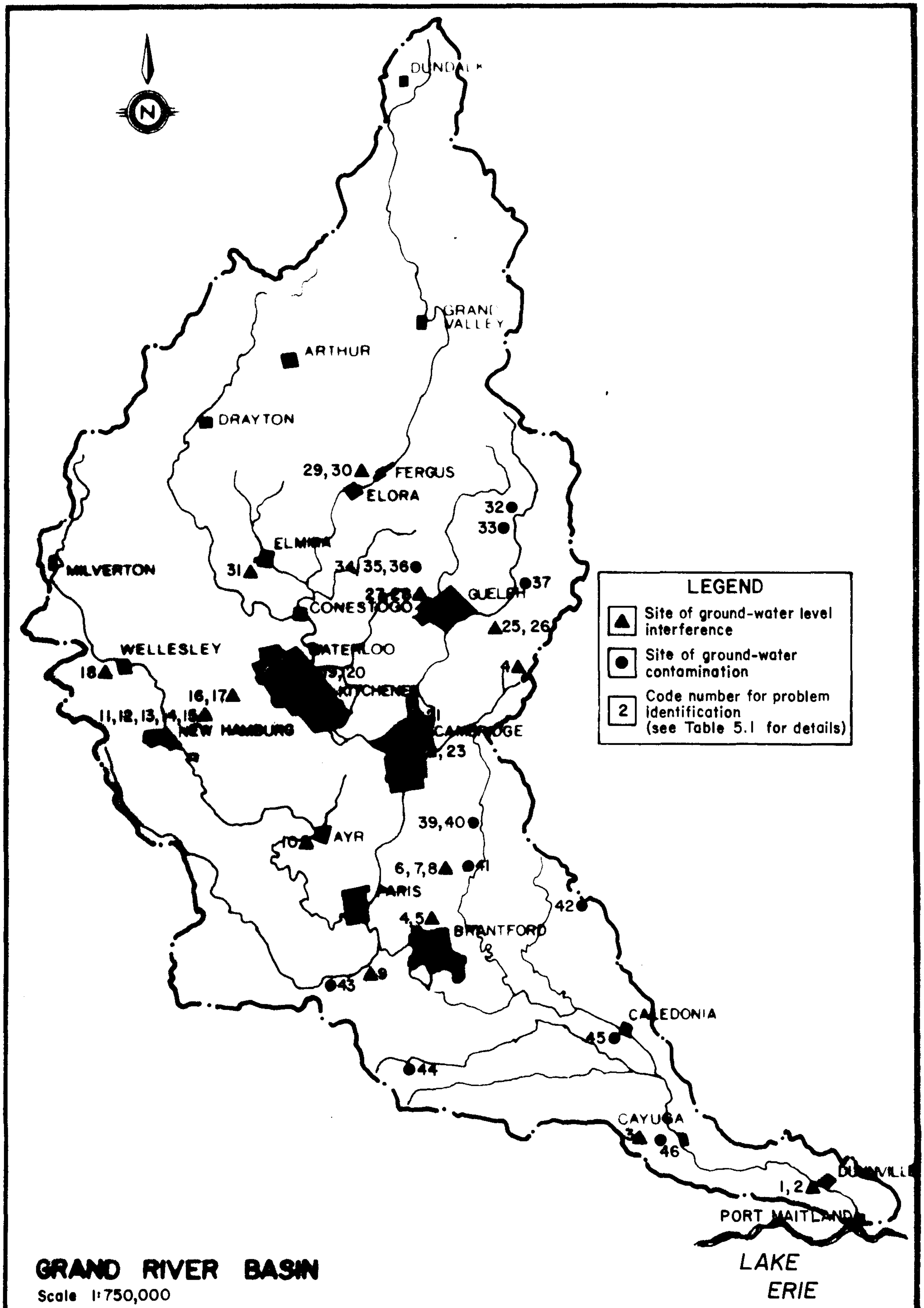


Figure 5.4. Locations of reported cases of ground-water level interference and contamination.

TABLE 5.1 Reported Cases of Ground-Water Level Interference and Contamination

A. Water Level Interference

Code Number	Area	Cause of Interference
1.	Town of Dunnville	Quarry Dewatering Operations
2.	Town of Dunnville	Water Main Construction
3.	Town of Cayuga	Quarry Expansion
4.	Twp. of Brantford	Municipal Well
5.	City of Brantford	Municipal and Commercial Wells
6.	Town of St. George	Culvert Installation
7.	Town of St. George	Industrial Well
8.	Town of St. George	Installation of Storm Sewers
9.	Twp. of Brantford	Highway Culvert Installation
10.	Village of Ayr	Bridge Construction
11.	Twp. of Wilmot	Municipal Wells
12.	Twp. of Wilmot	Municipal Wells
13.	Twp. of Wilmot	Municipal Wells
14.	Twp. of Wilmot	Municipal Wells
15.	Twp. of Wilmot	Municipal Wells
16.	Twp. of Wilmot	Municipal Wells
17.	Twp. of Wilmot	Municipal Wells
18.	Town of Wellesley	Flowing Wells
19.	City of Kitchener	Municipal Wells
20.	City of Waterloo	Municipal Wells
21.	City of Cambridge	Municipal Wells
22.	City of Cambridge	Municipal Wells
23.	City of Cambridge	Municipal Wells
24.	Twp. of Puslinch	Gravel Pit Dewatering
25.	Twp. of Puslinch	Municipal Wells
26.	City of Guelph	Municipal Wells
27.	City of Guelph	Construction of Sanitary Sewer

28.	City of Guelph	Municipal Wells
29.	Town of Fergus	Municipal Wells
30.	Twp. of W. Garafaxa	Municipal Well
31.	Town of Elmira	Municipal Well

B. Contamination

32.	Twp. of Eramosa	Fuel Oil Spill
33.	Twp. of Beverly	Salt Storage
34.	Twp. of Guelph	Road Salting
35.	Twp. of Pilkington	Fuel Oil Spill
36.	Twp. of Guelph	Road Salting
37.	Reg. Munic. of Halton	Fuel Oil Spill
38.	Twp. of Guelph	Industrial Waste Spill
39.	Sheffied	Stove Oil Spill
40.	Sheffied	Stove Oil Spill
41.	Twp. of S. Dumfries	Gasoline Spill
42.	Twp. of Ancaster	Road Salting
43.	Twp. of Burford	Gasoline Spill
44.	Wilsonville	Gasoline Spill
45.	Town of Caledonia	Bridge Construction
46.	Twp. of N. Cayuga	Gasoline Spill

responsible for restoration of the supplies. It is this condition in the Permits that has been used in the past to resolve complaints and restore supplies affected by large municipal takings. The restorations have usually involved replacement of supplies by deepening existing wells or constructing new ones.

Future issues of concern related to water supply interference will forseeably continue to be related to large-scale takings by municipalities, as well as to some large industrial and commercial uses. The expanding water supply needs of Kitchener-Waterloo, Cambridge and Guelph will necessitate ground-water exploration outside these municipal boundaries, placing pressure for ground-water development in more rural areas. Consequently, it is essential that in the course of future development of these water supplies, the potential for water-level interference with existing wells be recognized and appropriate allowances be made for the restoration of affected supplies. For example, the potential interference with nearby ground-water supplies and stream flow in the Galt Creek area should be recognized in the early planning for development of water supplies for Cambridge in the area. The same applies for the Roseville area if municipal supplies for Kitchener-Waterloo are to be developed from the aquifer identified in this report. On a more general scale, the potential for interference should be recognized prior to the development of municipal supplies in any of the candidate areas identified for test drilling in this report.

An issue of concern to local residents has been the development of an artificial ground-water recharge scheme near Mannheim. The scheme consists of recharging water from the Grand River in a gravel pit just east of Mannheim, and recovering the recharged water in numerous wells to be constructed for the purpose. Local concerns relate to possible effects produced by higher ground-water levels, such as possible flooding of land in low-lying areas adjacent to the scheme. However, if the recharge scheme is to be physically feasible and successful, part of that feasibility makes it manditory that these issues not become problems. For one, it is possible that

local hydrogeologic conditions in the area will be favourable to the recharge scheme without special measures being necessary to control water levels. Alternatively, the design and operation of the scheme will have to be flexible enough to allow controlling the rate of recharge and the recovery of water from the ground in order to control ground-water levels in the vicinity.

With a number of reservoirs proposed in the basin, the effects of these reservoirs on adjacent ground-water levels, and on possible changes in surface drainage, will need to be considered in the course of reservoir design. One obvious effect on ground-water levels might be to raise the water table locally as the reservoir is filled, and henceforth have ground-water levels fluctuate in response to changes in reservoir levels. The effects on ground water will likely be most pronounced in areas where sands and gravels are exposed to waters in the reservoir. Changes in ground-water levels in these areas can be expected to be rapid and to extend some distance back from the edge of the reservoir. High ground-water levels may subsequently create problems such as flooding of basements in houses and any number of other problems related to structural damages to buried pipelines, sewers, watermains and building foundations. The effects in areas of silt, clay and till can be expected to be negligible.

Other effects related to reservoirs may include flowing wells and flooding of land in some low-lying areas. All these potential effects depend on site-specific conditions of hydrogeology, reservoir design and existing and future land use practices, and many may be avoided by early awareness of potential problems and subsequent proper planning.

The chance of contaminating public ground-water supplies remains a concern with the major municipalities in the basin. To date there have been no documented cases of serious contamination of municipal supplies, but there are numerous cases of contamination of private domestic supplies. These cases have most frequently involved contamination by gasoline or highways deicing salts (Table 5.1). Contamination by gasoline has often been due to spills and to leaks from old storage tanks, while salt contamination has resulted from the application of deicing salts to highways which have subsequently infiltrated into the ground with snowmelt.

The prevention of similar, local contamination problems in the future is difficult because prevention will depend almost totally on individual efforts involving proper care, maintenance and operation of activities likely to result in ground-water contamination. On a broader scale, proper planning can be used in some cases to protect municipal aquifers from land use developments involving potential contaminating substances. These developments would include major highways, certain agricultural practices that involve spreading of fertilizers and feedlot operations, and a multitude of industrial and commercial developments involving liquid or solid materials that could pose a hazard. Information on Figure 5.3 may assist in avoiding contamination in sensitive areas.

As is obvious from the above discussions, there are a number of prominent environmental stresses placed on ground-water resources by development activities in both rural and urban settings. These stresses are usually the result of site-specific conditions of land use development and local hydrogeology, and as such, must be dealt with individually to minimize significant impacts on the resource. Handling of individual situations is almost imperative in most cases involving potential contamination of ground water, and should involve a thorough assessment of site-specific hydrogeologic conditions and their suitability for the development in question.

5.6 Long-Term Ground-Water Resources Potential in the Basin

In considering a plan for the integrated development of ground water for municipal uses in the basin, the question of long-term, total availability of water is basic. Obtaining ground water for small municipal demands is usually not an issue in most areas of the watershed, and as long as the demands remain small, ground-water supplies should be adequate for these communities on a long-term basis. However, as municipal demands grow, progressively larger ground-water takings are necessary, and ultimately these takings may begin to exceed the rate of natural recharge to ground water. Excess takings for short periods usually do not pose an immediate threat to the availability of ground water from an aquifer. However, if excess withdrawals continue for long periods, significant depletion of ground water may occur, leading in turn to possible interference with other surface and ground-water uses in the area.

To assist in the planning for long-term development of ground water for municipal uses by the years 2001 and 2031, the maximum amount of ground water potentially available for development in various parts of the basin must be addressed. If, for example, it becomes clear that water demands in 2001 or 2031 may exceed the maximum ground water available in the basin, then obviously long-term plans will have to consider alternate sources of supply for municipal uses. If, on the other hand, the maximum available supply can be shown to exceed projected demands, plans for development of this water on a long-term basis can be considered now.

The long-term, maximum amount of ground water potentially available for development in the basin cannot exceed the total recharge to ground water from precipitation. Additionally, since some of this recharge is needed to maintain streamflow during summer months, only a fraction of the total recharge to ground water is available for development by municipal wells. For the purposes of present estimates of ground water availability, the assumption has been made that a maximum of 50% of the total recharge to ground water may be available for development.

Estimates of ground-water recharge rates were based on analyses of streamflows at 15 stations in the basin (Figure 5.5, Table 5.2). These analyses consisted of obtaining monthly minimum streamflows and averaging these for each year of available data for the years 1970-1977 (Appendix I). The yearly means were then averaged to arrive at one streamflow value. Only natural, unregulated flows were used whenever possible; deregulation of flows was necessary in some streams in the basin. The estimates obtained by this method are probably conservative because only movement of shallow ground water is considered and no account is made for deep percolation of water that must take place to recharge deeper aquifers. However, the estimates are considered suitable for the purpose of obtaining first approximations necessary at this stage to compare infiltration rates throughout the basin and to indicate areas with relatively high values.

Estimates of infiltration rates for nine individual basins with unregulated flows are shown on Figure 5.5. In comparing these rates with surficial geology, the least permeable areas covered by silt, clay and till have the lowest rates, varying from about 0.07 mgd per square mile in the till area upstream of the Nithburg gauge (A38), to about 0.13 mgd per square mile in the predominantly clay area of Mackenzie Creek drainage (upstream of gauge B10). The largest values of 0.25 and 0.24 were obtained for the Eramosa River upstream of Guelph (A29) and for Whitemans Creek (B8), respectively. Both sub-basins contain large areas of permeable sands, gravels and/or limestone bedrock at or near the surface. Estimates for primarily sand and gravel areas could not be obtained. However, based on the higher infiltration rates, it is likely that infiltration in areas of primarily surficial sands, gravels, and/or limestone can be considerably higher than 0.25 mgd per square mile. Therefore, infiltration rates higher than 0.25 are likely in areas such as in the vicinity of Guelph, west and southwest of Kitchener-Waterloo, and around Cambridge, where permeable surficial materials predominate.

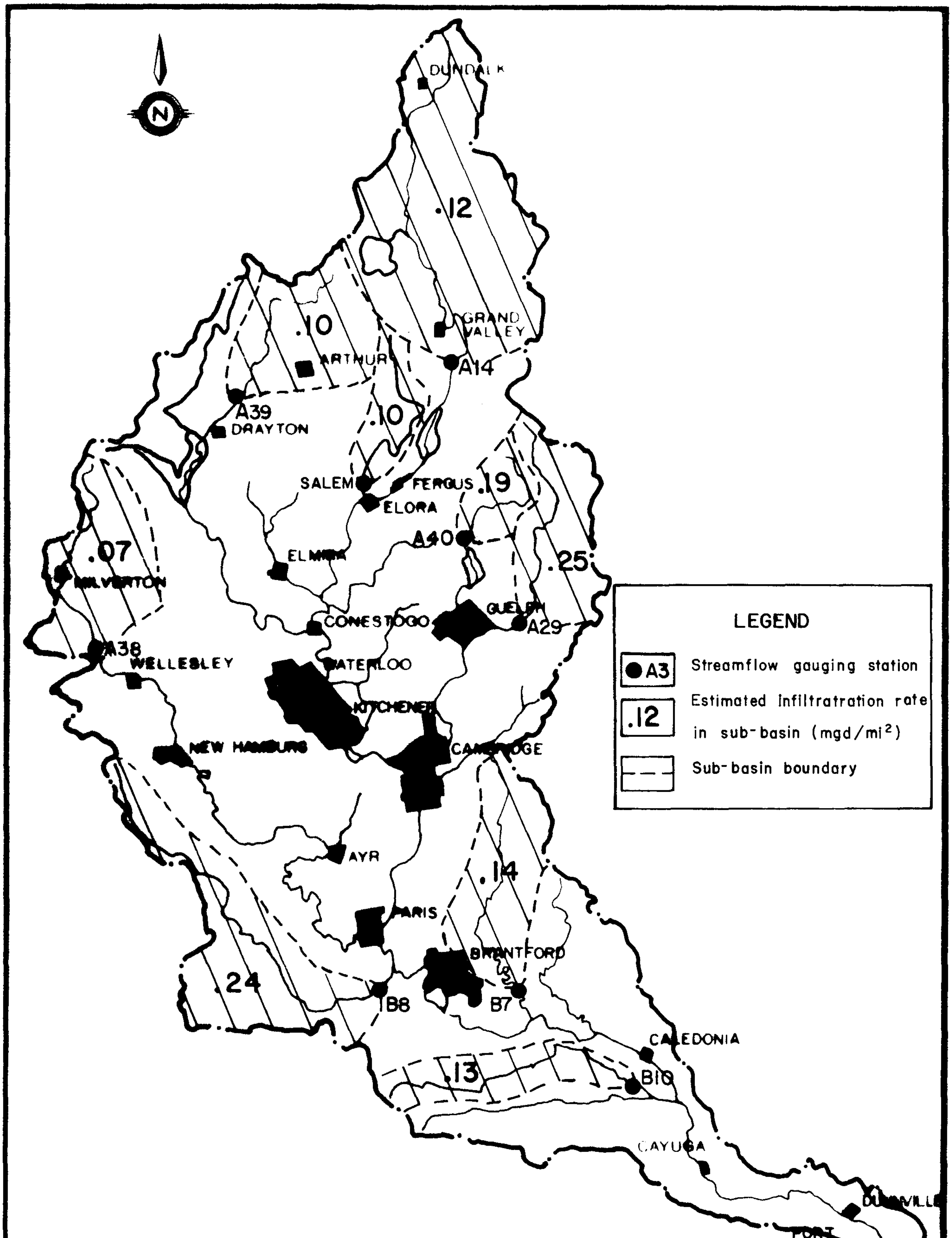


Figure 5.5. Approximate ground-water infiltration rates in nine sub-basins.

TABLE 5.2

A. Estimates of Ground-Water Infiltration Rates In Nine Sub-Basins in the Grand River Basin (Shown on Figure 5.5)

Gauging Station	Sub-Basin	Predominant Surficial Material	Drainage Area (mi^2)	Baseflow (mgd)	Infiltration Rate (mgd/mi^2)
A14	Grand R. upstream of Marsville	Till	268	31.6	.12
A39	Conestogo R. upstream of Drayton	Till	105	10.9	.10
SALEM	Irvine Cr. upstream of Salem	Till	84	8.06	.10
A40	Speed R. upstream of Armstrong Mills	Variable: Till, Sand, Gravel, Lms.	64	12.5	.19
A29	Eramosa R. upstream of Guelph	Variable: Till, Sand, Gravel, Lms.	91	23.2	.25
A38	Nith R. upstream of Nithburg	Till	126	8.27	.07
B8	Whitemans Cr.	Variable: Till, Sand, Gravel	148	35.5	.24
B7	Fairchild Cr.	Variable: Till, Clay Sand, Gravel	139	19.8	.14
B10	McKenzie Cr.	Clay	66	8.35	.13

TABLE 5.2

B. Estimates of Ground-Water Infiltration Rates in Six Sub-Basins in the Grand River Basin (Not shown on Figure 5.5)

TABLE 5.2

B. Estimates of Ground-Water Infiltration Rates in Six Sub-Basins in the Grand River Basin (Not shown on Figure 5.5)

Gauging Station	Sub-Basin	Predominant Surficial Material	Drainage Area (mi^2)	Baseflow (mgd)	Infiltration Rate (mgd/mi^2)
DOON	Grand R. upstream of Doon	Variable: Till, Sand, Gravel	931	154	0.17
A3	Grand R. upstream of Galt	Variable: Till, sand, Gravel	1360	220	0.16
B1	Grand R. upstream of Brantford	Variable: Till, Sand, Gravel	2010	390	0.19
A18	Nith R. upstream of N. Hamburg	Till	213	22.2	0.10
A10	Nith R. upstream of Canning	Variable: Till, Sand Gravel	398	79.5	0.20
A15	Speed R. upstream of Guelph	Variable: Till, Sand, Gravel, Lms	229	46.4	0.20

For purposes of planning only, the surficial clay area south of Brantford and the till plain areas in the northern part of the basin should not be considered suitable for the development of long-term, large municipal supplies. This leaves the central area of the basin, where infiltration rates in the sand and gravel areas are considerably higher, for the potential development of large-capacity supplies. Fortunately, this is also where the large demands for municipal supplies are located.

The amount of ground water potentially available in the central area of the basin can be approximated by the baseflow estimates shown in Table 5.2. Subtracting the combined baseflow upstream of Marsville, Drayton, New Hamburg, Salem and Whitemans Creek (108 mgd) from that at Brantford (390 mgd), yields approximately 282 mgd in the generally central area of the basin. The projected 2001 and 2031 average day demands by municipalities in the central part of the basin (Kitchener-Waterloo, Cambridge, Guelph, Paris, St. Jacobs, New Hamburg-Baden, Fergus and Elora) are estimated to be 68 mgd and 103 mgd, respectively. Assuming 50% of the 282 mgd is the maximum potentially available, the potential supply is more than twice the projected demand in 2001, and 37% greater than the demand in 2031. However, this comparison is primarily academic because it would not be possible to recover the 141 mgd over the 1192 square mile area in which it infiltrates.

The foregoing estimates are meant only to place ground water in a general perspective relating availability to projected demands. The estimates can not, and should not, be used in any way to plan and develop individual municipal wells. It is likely that individual situations may allow considerably greater extractions of ground water to satisfy municipal needs past 2001 and 2031. Consequently, it is only site-specific determinations of long-term well yields that should be used to guide the planning and development of ground water for specific municipal supplies as needs arise.

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APPENDIX A

GEOLOGIC AND GEOPHYSICAL LOGS OF MOE TEST HOLES

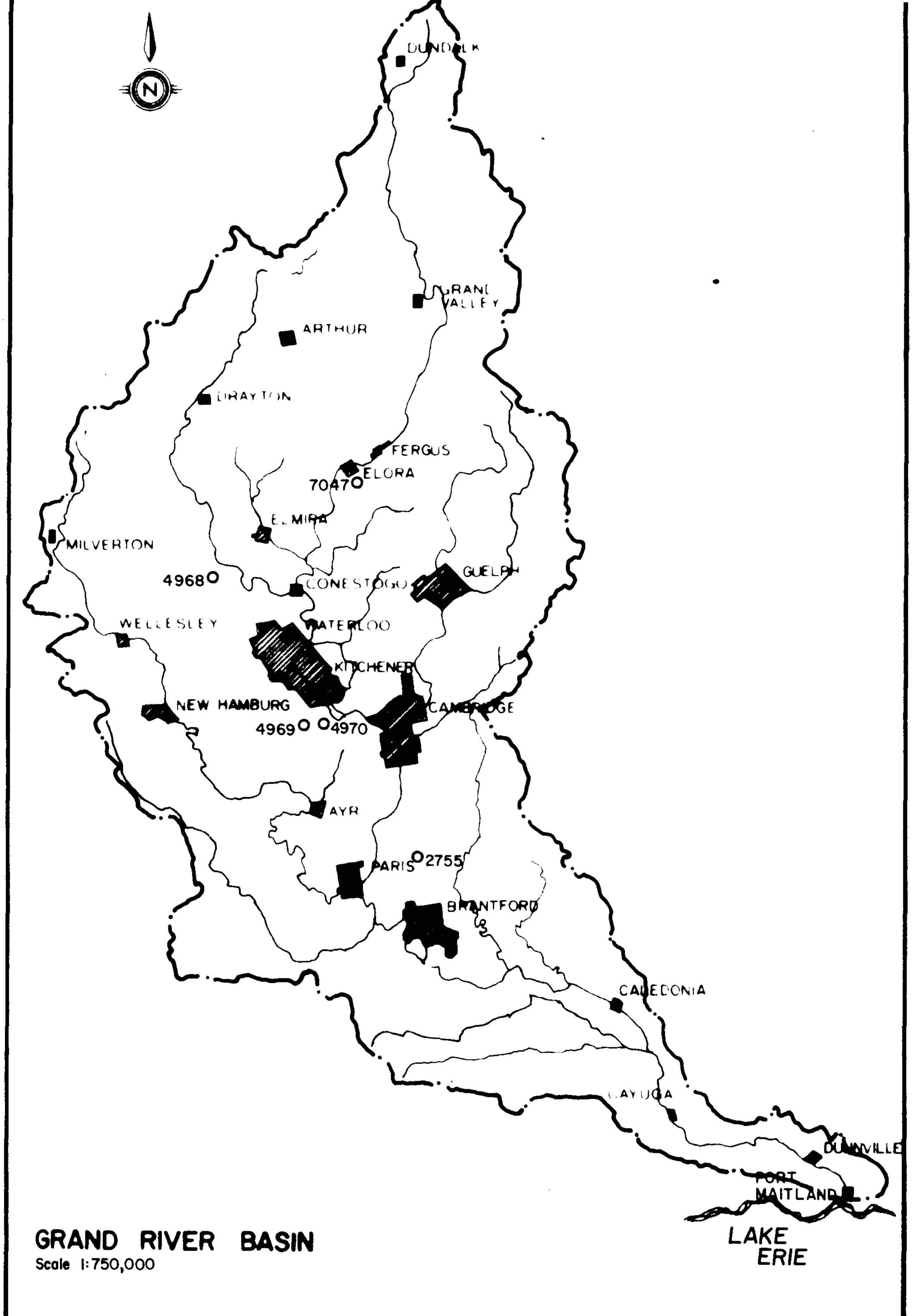
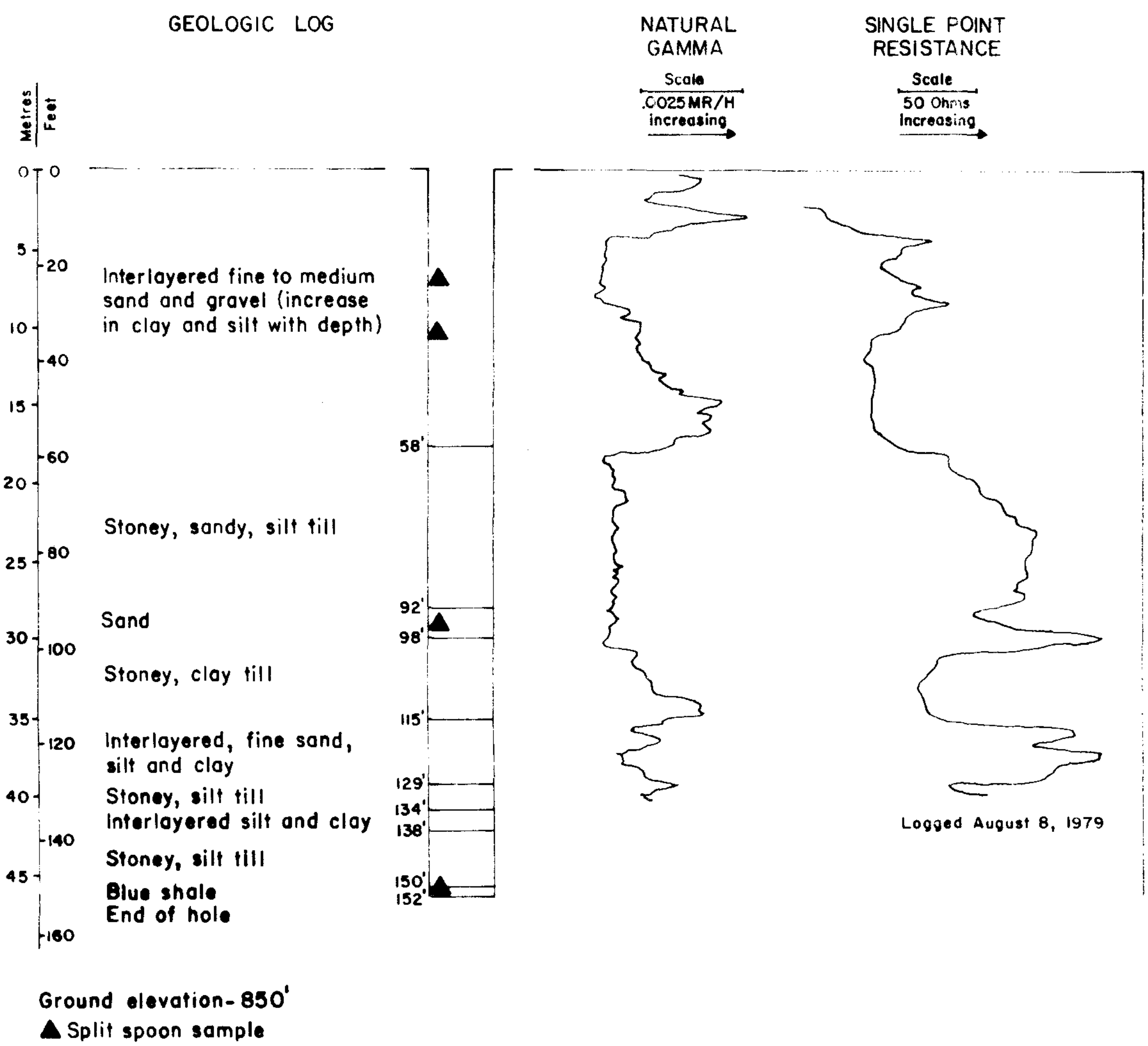


Figure A.1. Locations of Ministry of the Environment test holes with respective well numbers (records on file with M.O.E.).


GEOLOGIC LOG
NATURAL GAMMA
Scale
.0025 MR/H increasing
SINGLE POINT RESISTANCE
Scale
50 Ohms Increasing
Metres
Feet
0
5
10
15
20
25
30
35
40
45
0
20
40
60
80
100
120
140
160
Interlayered fine to medium sand and gravel (increase in clay and silt with depth)
58'
Stoney, sandy, silt till
92'
Sand
98'
Stoney, clay till
115'
Interlayered, fine sand, silt and clay
129'
Stoney, silt till
134'
Interlayered silt and clay
138'
Stoney, silt till
150'
Blue shale
152'
End of hole
Logged August 8, 1979
Ground elevation- 850'
Split spoon sample

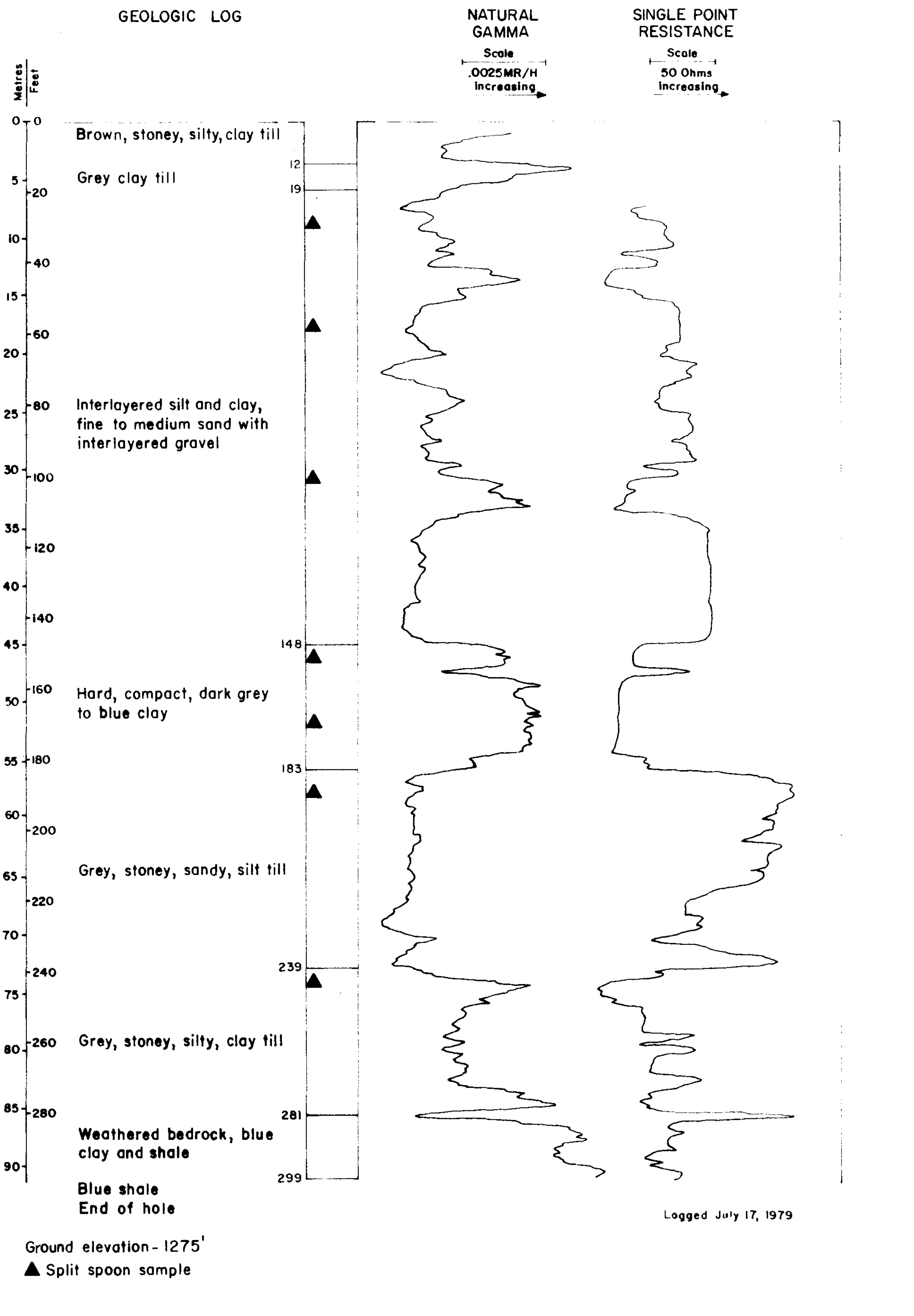
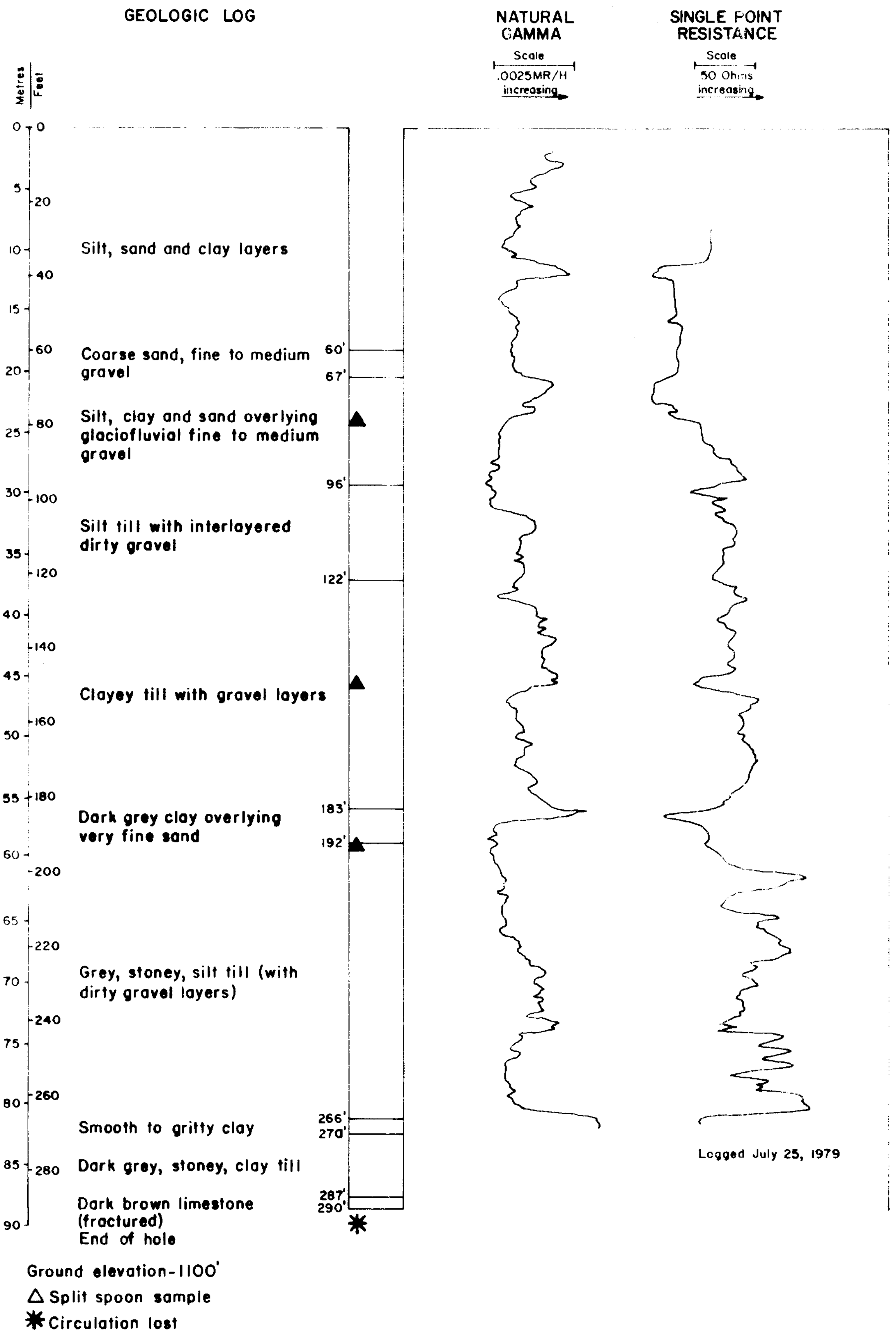


Figure A.3. Geologic and geophysical well log of Test Hole 4968 at St. Clements.



GEOLOGIC LOG
NATURAL GAMMA
Scale
.0025MR/H
increasing
SINGLE POINT RESISTANCE
Scale
50 Ohms
increasing
Metres
Feet
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280
Silt, sand and clay layers
60'
Coarse sand, fine to medium gravel
67'
Silt, clay and sand overlying glaciofluvial fine to medium gravel
96'
Silt till with interlayered dirty gravel
122'
Clayey till with gravel layers
183'
Dark grey clay overlying very fine sand
192'
Grey, stoney, silt till (with dirty gravel layers)
266'
Smooth to gritty clay
270'
Dark grey, stoney, clay till
287'
Dark brown limestone (fractured)
290'
End of hole
Ground elevation-1100'
Split spoon sample
Circulation lost
Logged July 25, 1979

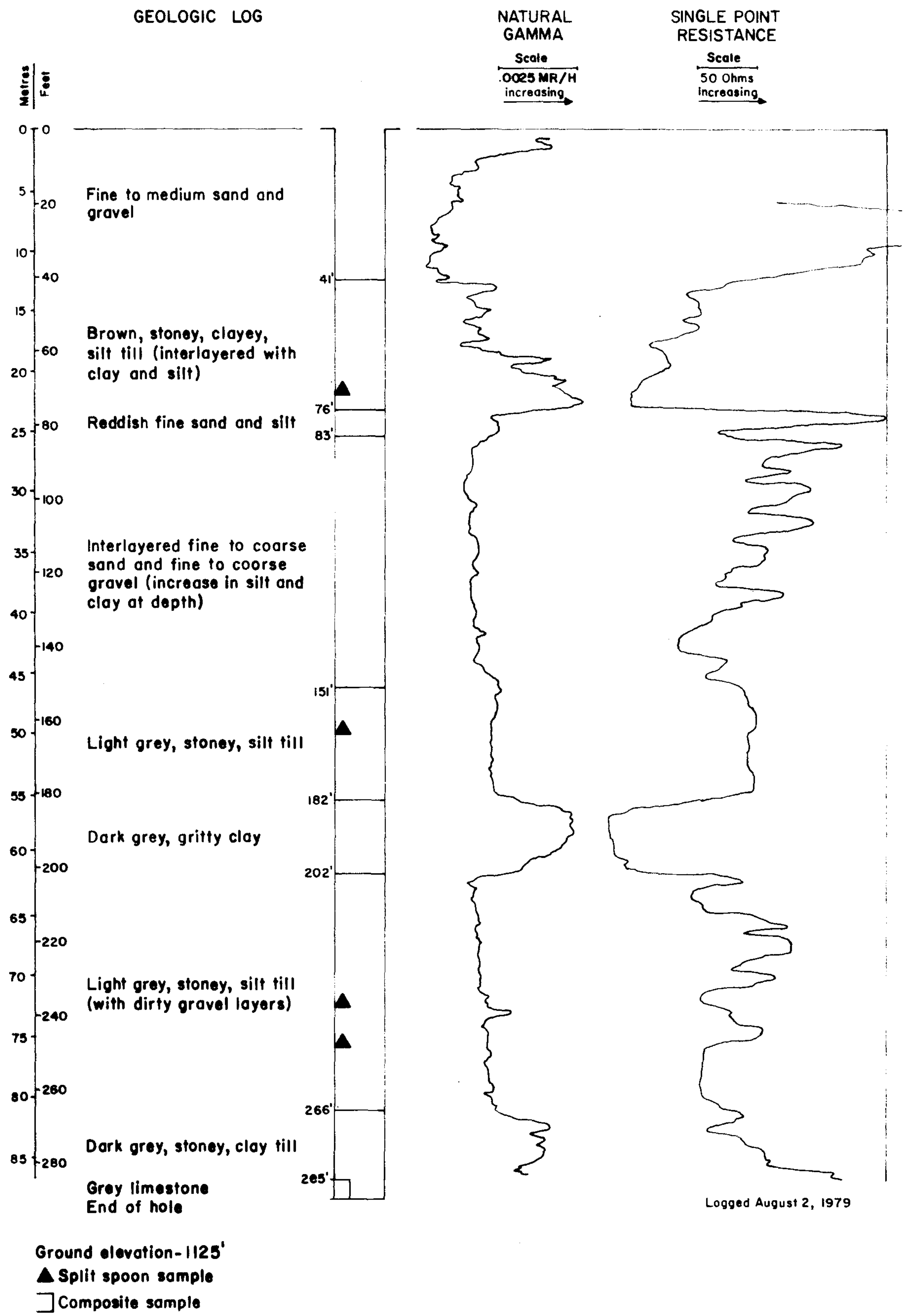
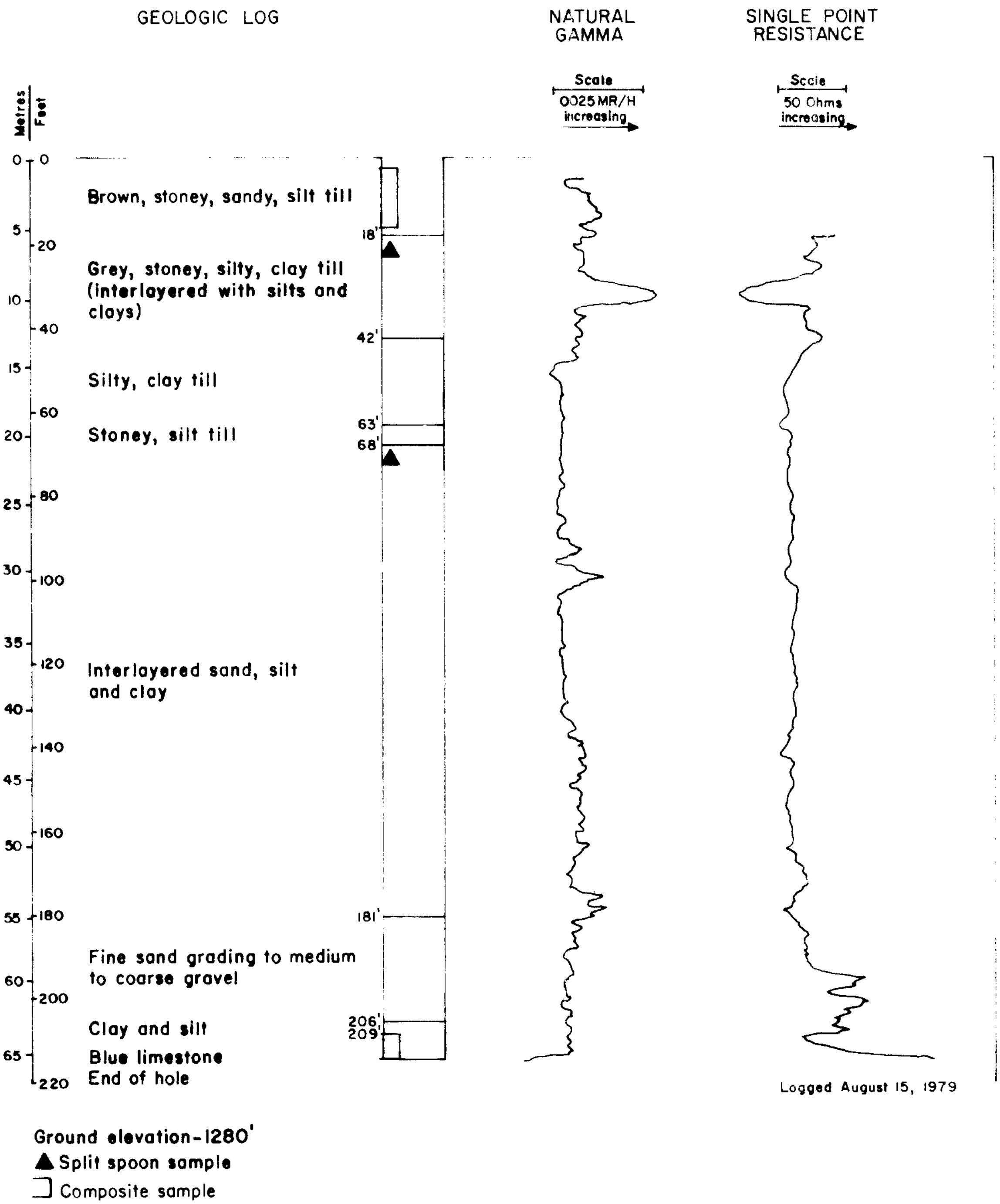


Figure A.5. Geologic and geophysical well log of Test Hole 4970 at Roseville.



GEOLOGIC LOG
NATURAL GAMMA
SINGLE POINT RESISTANCE
Scale
0025 MR/H increasing
Scale
50 Ohms increasing
Metres
Feet
0
5
10
15
20
25
30
35
40
45
50
55
60
65
0
20
40
60
80
100
120
140
160
180
200
220
Brown, stoney, sandy, silt till
18'
Grey, stoney, silty, clay till (interlayered with silts and clays)
42'
Silty, clay till
63'
Stoney, silt till
68'
Interlayered sand, silt and clay
181'
Fine sand grading to medium to coarse gravel
206'
209'
Clay and silt
Blue limestone
End of hole
Logged August 15, 1979
Ground elevation-1280'
Split spoon sample
Composite sample

APPENDIX B

POPULATION PROJECTIONS FOR INCORPORATED MUNICIPALITIES IN THE GRAND RIVER BASIN, 1976 - 2031

		1976	1981	1986	1991	1996	2001	2006	2011	2016	2021	2026	2031
ARTHUR	*LOW	1623	1710	1772	1871	1758	2038	2120	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	*MED	0	0	0	0	0	0	0	0	0	0	0	0
	*HIGH	1578	1759	1937	2035	2193	2352	2544	2741	2953	3131	3427	3672
AYR	*LOW	1331	1543	1799	2017	2083	2143	2227	2237	2334	2353	2367	2377
	*MED	1331	1547	1779	2076	2110	2283	2452	2571	2789	2958	3123	3297
	*HIGH	1331	1547	1803	2100	2167	2376	2586	2802	3027	3270	3521	3769
BRANTFORD	*LOW	69730	73497	77246	81187	85323	89630	94245	99255	104115	109427	115609	120875
	*MED	69730	75001	80440	86273	92529	99239	106436	114154	122432	131310	140832	151045
	*HIGH	69730	76530	83757	91656	100346	109772	120131	131459	143876	157434	172313	188575
CALEDONIA	*LOW	3669	4119	4612	5171	5797	6500	7296	8189	9192	10313	11582	13000
	*MED	0	0	0	0	0	0	0	0	0	0	0	0
	*HIGH	3669	4752	6036	7833	10874	13009	13846	14661	15570	16536	17560	18649
CAMBRIDGE	*LOW	71492	83412	92623	100702	108612	116015	121412	124719	127754	128770	129032	129029
	*MED	71482	83463	92649	103882	114179	124874	133602	142983	152063	161270	170293	178462
	*HIGH	71482	83463	92838	105069	114932	129525	141011	152759	165058	178270	191967	205497
CAYUGA	*LOW	1143	1154	1165	1177	1189	1200	1356	1533	1732	1957	2212	2500
	*MED	0	0	0	0	0	0	0	0	0	0	0	0
	*HIGH	1143	1337	1563	1828	2138	2500	2655	2820	2995	3179	3377	3586
DRAYTON	*LOW	813	854	898	943	992	1042	1095	1152	1210	1272	1337	1405
	*MED	813	875	944	1016	1075	1179	1271	1369	1475	1590	1712	1843
	*HIGH	813	919	1040	1177	1476	1587	1793	1929	2182	2470	2796	3162
DUNDALK	*LOW	1130	1172	1270	1372	1470	1561	1658	1760	1869	1995	2103	2238
	*MED	1130	1247	1377	1521	1679	1854	2047	2260	2495	2755	3041	3358
	*HIGH	1130	1278	1446	1637	1852	2094	2370	2682	3034	3433	3884	4394
DUNNVILLE	*LOW	5430	5554	5833	6937	6762	7183	7961	7838	8234	8650	9087	9545
	*MED	5430	5587	5952	6339	6749	7187	7652	8149	8670	9249	9840	10470
	*HIGH	5430	5626	6074	6556	7077	7639	8246	8900	9607	10369	11193	12082
ELMIRA	*LOW	7034	7573	8028	8313	8549	8781	9044	9270	9479	9555	9611	9574
	*MED	7034	7578	8822	8559	8967	9272	9935	10644	11327	12015	12485	13302
	*HIGH	7034	7578	8038	8657	9180	9648	10594	11379	12295	13288	14299	15387
ELORA	*LOW	2979	3243	3537	3858	4208	4589	5005	5458	5953	6492	7080	7722
	*MED	2974	3376	3763	4232	4760	5355	6025	6774	7620	8571	9640	10844
	*HIGH	2974	3440	3997	4633	5371	6227	7219	8368	9701	11246	13038	15114
FERGUS	*LOW	5967	6457	6969	7481	7993	8505	9017	9529	10041	10553	11065	11577
	*MED	5967	6901	7777	8765	9878	11133	12546	14139	15935	17959	20240	22810
	*HIGH	5967	7381	8464	9812	11375	13187	15288	17723	20545	23818	27611	32009
GRAND VALLEY	*LOW	1060	1141	1223	1322	1423	1537	1649	1775	1910	2057	2214	2383
	*MED	1060	1170	1292	1427	1575	1739	1920	2120	2341	2584	2853	3150
	*HIGH	1060	1213	1482	1612	1853	2132	2457	2819	3242	3729	4287	4931
GUELPH	*LOW	70374	73964	77737	81702	85870	90250	94854	99692	104777	110122	115739	121643
	*MED	70374	77697	85783	94714	104572	115456	127473	140746	155389	171562	189418	209132
	*HIGH	70374	79622	90085	101923	115316	130469	147614	167012	188958	213789	241893	273662
KITCHENER	*LOW	131291	147295	166656	181382	194243	208795	217812	227714	229970	229270	238632	229278
	*MED	131801	147304	166694	186456	205160	222486	238072	255474	271793	288306	304307	319348
	*HIGH	131801	147305	167070	188782	210199	231510	252813	273807	295826	318690	343193	367581
MILVERTON	*LOW	1402	1424	1446	1469	1492	1516	1540	1564	1588	1613	1639	1664
	*MED	1402	1519	1634	1764	1905	2057	2221	2398	2589	2794	3017	3255
	*HIGH	1402	1625	1884	2184	2532	2935	3403	3945	4573	5301	6146	7125
NEW HAMBURG	*LOW	3623	3737	4174	4237	4376	4535	4713	4841	4963	4979	5087	4999
	*MED	3623	3941	4175	4362	4628	4837	5108	5547	5983	6261	6611	6935
	*HIGH	3623	3941	4193	4412	4733	5029	5474	5938	6487	6921	7452	7977
PARIS	*LOW	6671	7620	7714	7800	7904	8000	8049	8192	9229	8581	8564	8602
	*MED	6691	7620	7956	8139	8437	8850	9123	9457	9803	10163	10535	10921
	*HIGH	6671	7620	8043	8501	8979	9484	10017	10580	11175	11803	12467	13168
WATERLOO	*LOW	49972	58916	68828	74772	80597	86461	89044	92333	94130	94940	95584	95128
	*MED	49972	58954	68836	76930	84691	92131	98916	105770	112551	119306	126044	132237
	*HIGH	49972	58954	68970	77860	86771	95869	104370	113066	122169	131958	142086	152097
WELLESLEY	*LOW	842	997	1179	1344	1369	1461	1519	1560	1592	1605	1614	1608
	*MED	842	998	1180	1304	1439	1557	1672	1788	1902	2018	2130	2235
	*HIGH	842	998	1182	1400	1474	1620	1764	1911	2065	2230	2401	2571

<u>APPENDIX C</u>

POPULATION PROJECTIONS FOR
UNINCORPORATED COMMUNITIES IN THE
GRAND RIVER BASIN, 1976 - 2031

Year	Projection 1 (Township of Wilmot)	Baden	Projection 2 (Township of Wilmot)	Baden	Projection 3 (Township of Wilmot)	Baden
1976	10,566	824	10,566	824	10,566	824
1981	11,592	904	11,592	904	11,585	904
1986	12,302	960	12,279	958	12,276	958
1991	12,977	1,012	12,830	1,001	12,462	972
1996	13,920	1,086	13,587	1,060	12,930	1,009
2001	14,788	1,153	14,212	1,109	13,337	1,040
2006	16,100	1,256	15,258	1,190	13,862	1,081
2011	17,441	1,360	16,316	1,273	14,239	1,111
2016	18,845	1,470	17,362	1,354	14,529	1,133
2021	20,355	1,588	18,416	1,436	14,645	1,142
2026	21,918	1,710	19,443	1,517	14,732	1,149
2031	23,462	1,830	20,398	1,591	14,674	1,145

Note: In 1976, the community of Baden contained 7.8% of the Wilmot Township population. It was assumed that the community would contain this proportion of the population to the year 2031.

Community of Burford

Year	Projection 1 (Township of Burford)	Burford	Projection 2 (Township of Burford)	Burford	Projection 3 (Township of Burford)	Burford
1971	2,996		2,996		2,996	
1976	2,899	1,051	2,899	1,051	2,899	1,051
1981	3,084	1,118	2,972	1,077	2,884	1,046
1986	3,202	1,161	3,047	1,105	2,870	1,040
1991	3,478	1,261	3,124	1,133	2,855	1,035
1996	3,537	1,282	3,203	1,161	2,804	1,029
2001	3,718	1,348	3,284	1,191	2,826	1,025
2006	3,907	1,416	3,367	1,221	2,811	1,019
2011	4,107	1,489	3,452	1,251	2,796	1,014
2016	4,316	1,565	3,539	1,283	2,781	1,008
2021	4,536	1,644	3,628	1,315	2,766	1,003
2026	4,68	1,729	3,720	1,349	2,750	997
2031	5,011	1,817	3,814	1,383	2,735	992

Note: Population projections for the Village of Burford were derived by assuming that Burford retains the present proportion (36.25%) of the Township's population in the future.

Community of Maryhill

Year	Projection 1 (Township of Woolwich)	Maryhill	Projection 2 (Township of Woolwich)	Maryhill	Projection 3 (Township of Woolwich)	Maryhill
1976	16,038	409	16,038	409	16,038	409
1981	17,223	448	17,223	448	17,212	448
1986	18,268	475	18,232	474	18,228	474
1991	19,674	512	19,452	506	18,894	491
1996	20,881	543	20,380	530	19,395	504
2001	21,928	570	21,072	548	19,776	514
2006	23,872	621	22,624	588	20,554	534
2011	25,861	672	24,192	629	21,113	549
2016	27,943	727	25,743	669	21,543	560
2021	30,181	785	27,306	710	21,715	565
2026	32,498	845	28,829	750	21,844	568
2031	34,788	904	30,246	786	21,758	566

Note: In 1976, the community of Maryhill contained 2.6% of the Woolwich Township population. It was assumed that the community would contain this proportion of the population to the year 2031.

Community of Plattsville

Year	Projection 1 (Township of Blandford-Blenheim)	Plattsville	Projection 2 (Township of Blandford-Blenheim)	Plattsville	Projection 3 (Township of Blandford-Blenheim)	Plattsville
1976	5,959	498	5,959	498	5,959	498
1981	6,420	536	6,263	523	6,109	511
1986	6,916	578	6,582	550	6,264	523
1991	7,450	623	6,918	578	6,422	537
1996	8,026	671	7,271	608	6,584	550
2001	8,646	723	7,642	639	6,750	564
2006	9,314	778	8,032	671	6,921	578
2011	10,034	839	8,442	706	7,096	593
2016	10,810	903	8,872	741	7,275	608
2021	11,645	973	9,325	779	7,458	623
2026	12,545	1,048	9,800	819	7,647	639
2031	13,515	1,129	10,300	861	7,840	655

Note: Approximately 8.35% of the Township population (within the Grand River watershed) resides in Plattsville. It was assumed that this proportion would remain constant to 2031.

Community of Rockwood

Year	Projection 1 (Township of Eramosa)	Rockwood	Projection 2 (Township of Eramosa)	Rockwood	Projection 3 (Township of Eramosa)	Rockwood
1976	4,084	939	4,084	939	4,084	939
1981	4,689	1,172	4,509	1,172	4,292	1,172
1986	5,383	1,453	4,978	1,344	4,511	1,218
1991	6,180	1,675	5,497	1,490	4,741	1,285
1996	7,095	1,924	6,068	1,645	4,983	1,351
2001	8,145	2,208	6,700	1,817	5,237	1,420
2006	9,351	2,535	7,398	2,006	5,505	1,493
2011	10,736	2,911	8,168	2,216	5,785	1,568
2016	12,326	3,342	9,018	2,445	6,081	1,649
2021	14,151	3,837	9,956	2,699	6,391	1,733
2026	16,246	4,405	10,992	2,980	6,717	1,821
2031	18,651	5,057	12,137	3,290	7,059	1,914

Note: The community of Rockwood contained approximately 23% of the total Township population in 1976. This percentage is expected to increase slightly. By 1990, if municipal services are installed, the population is expected to reach 1630. This figure represents 27.11% of the total projected high population of the Township. Therefore, it was assumed that the community population relative to the Township population would increase from 23% to 27% and remain constant between 1991 and 2031.

Community of Salem

Year	Projection 1 (Township of Nichol)	Salem	Projection 2 (Township of Nichol)	Salem	Projection 3 (Township of Nichol)	Salem
1971	2,125		2,125		2,125	
1976	2,766	743	2,766	743	2,766	743
1981	3,054	820	2,907	781	2,836	762
1986	3,372	906	3,055	821	2,907	781
1991	3,723	1,000	3,211	863	2,981	801
1996	4,110	1,104	3,375	907	3,056	821
2001	4,538	1,219	3,547	953	3,133	841
2006	5,010	1,346	3,728	1,001	3,212	863
2011	5,532	1,486	3,918	1,052	3,293	885
2016	6,107	1,641	4,118	1,106	3,377	907
2021	6,743	1,811	4,328	1,163	3,462	930
2026	7,445	2,000	4,549	1,222	3,549	953
2031	8,220	2,208	4,781	1,284	3,639	978

Note: Population projections for the Village of Salem were derived by assuming that Salem retains the present proportion (26.86%) of the Township's population in the future.

Community of St. George

Year	Projection 1 (Township of S. Dumfries)	St. George	Projection 2 (Township of S. Dumfries)	St. George	Projection 3 (Township of S. Dumfries	St. George
1976	3,951	980	3,951	930	3,951	930
1981	4,153	1,083	4,051	1,056	4,000	1,043
1986	4,364	1,262	4,153	1,200	4,051	1,171
1991	4,587	1,470	4,258	1,365	4,102	1,315
1996	4,821	1,712	4,365	1,550	4,153	1,475
2001	5,067	2,000	4,476	1,766	4,205	1,659
2006	5,325	2,330	4,589	2,008	4,258	1,863
2011	5,597	2,714	4,705	2,281	4,312	2,091
2016	5,883	3,162	4,823	2,592	4,366	2,347
2021	6,183	3,683	4,945	2,946	4,421	2,633
2026	6,498	4,290	5,070	3,347	4,476	2,955
2031	6,829	4,997	5,198	3,803	4,533	3,317

Note: It was assumed that if the Community were to grow at a rate of 3.1% per annum the ultimate population would be reached by the year 2001. The high projection for the community of St. George was derived by assuming a 3.1% per annum population increase. The medium and low population projections were derived by assuming that the population would grow in the same proportion relative to the Township population as was presented for the high projection.

Community of St. Jacobs

Year	Projection 1 (Township of Woolwich)	St. Jacobs	Projection 2 (Township of Woolwich)	St. Jacobs	Projection 3 (Township of Woolwich)	St. Jacobs
1976	16,038	852	16,038	852	16,038	852
1981	17,223	913	17,223	913	17,212	912
1986	18,268	968	18,232	966	18,228	966
1991	19,674	1,043	19,452	1,031	18,894	1,001
1996	20,881	1,107	20,380	1,080	19,395	1,028
2001	21,928	1,162	21,072	1,117	19,776	1,048
2006	23,872	1,265	22,624	1,199	20,554	1,089
2011	25,861	1,371	24,192	1,282	21,113	1,119
2016	27,943	1,481	25,743	1,364	21,543	1,142
2021	30,181	1,600	27,306	1,447	21,715	1,151
2026	32,498	1,722	28,829	1,528	21,844	1,158
2031	34,788	1,844	30,246	1,603	21,758	1,153

Note: In 1976, the community of St. Jacobs contained 5.3% of the Woolwich Township population. It was assumed that the community would contain this proportion of the population to the year 2031.

APPENDIX D

CAMBRIDGE GROUND-WATER
DEVELOPMENT PROJECT

CAMBRIDGE GROUND-WATER DEVELOPMENT PROJECT

A. Description

A three year, $225,000 ground-water study has been undertaken by the Regional Municipality of Waterloo in the Cambridge area. The purpose of this study is to determine additional sources of ground water within a reasonable distance of Cambridge. Exploration is being undertaken both within the Regional territory and in the townships of Puslinch and South Dumfries. These townships are located in the counties of Wellington and Brant, respectively.

The geology of the areas under review is described in the "Interim Summary Report" 1978. While the results of this study will not be available until late 1980, it has been (conservatively) estimated that 5-7 mgd can be developed in the study area. With the "Interim Summary Report" as reference, the following sites (figures D.1, D.2) and associated developed capacities are utilized to develop project costs:

Site	Capacity (mgd)
Deans Tract	1.440
Orrs Lake	.504
Galt Sportman's Club	1.008
Bond Tract	1.008
Paddock Farm	.720
University of Guelph	1.440
	6.120

B. Design Criteria

It is assumed that this group of wells, which will include both overburden and rock types, will be equipped with vertical turbine pumps and hollow-shaft electric motors. A 12 x 16-foot insulated pump-house of prepainted, prefabricated steel will be provided. Pump-house equipment will include a reduce-voltage starter, turbine meter, flow control and isolating valves, air relief valve, 15 KW heater and supervisory controls to provide for remote operation.

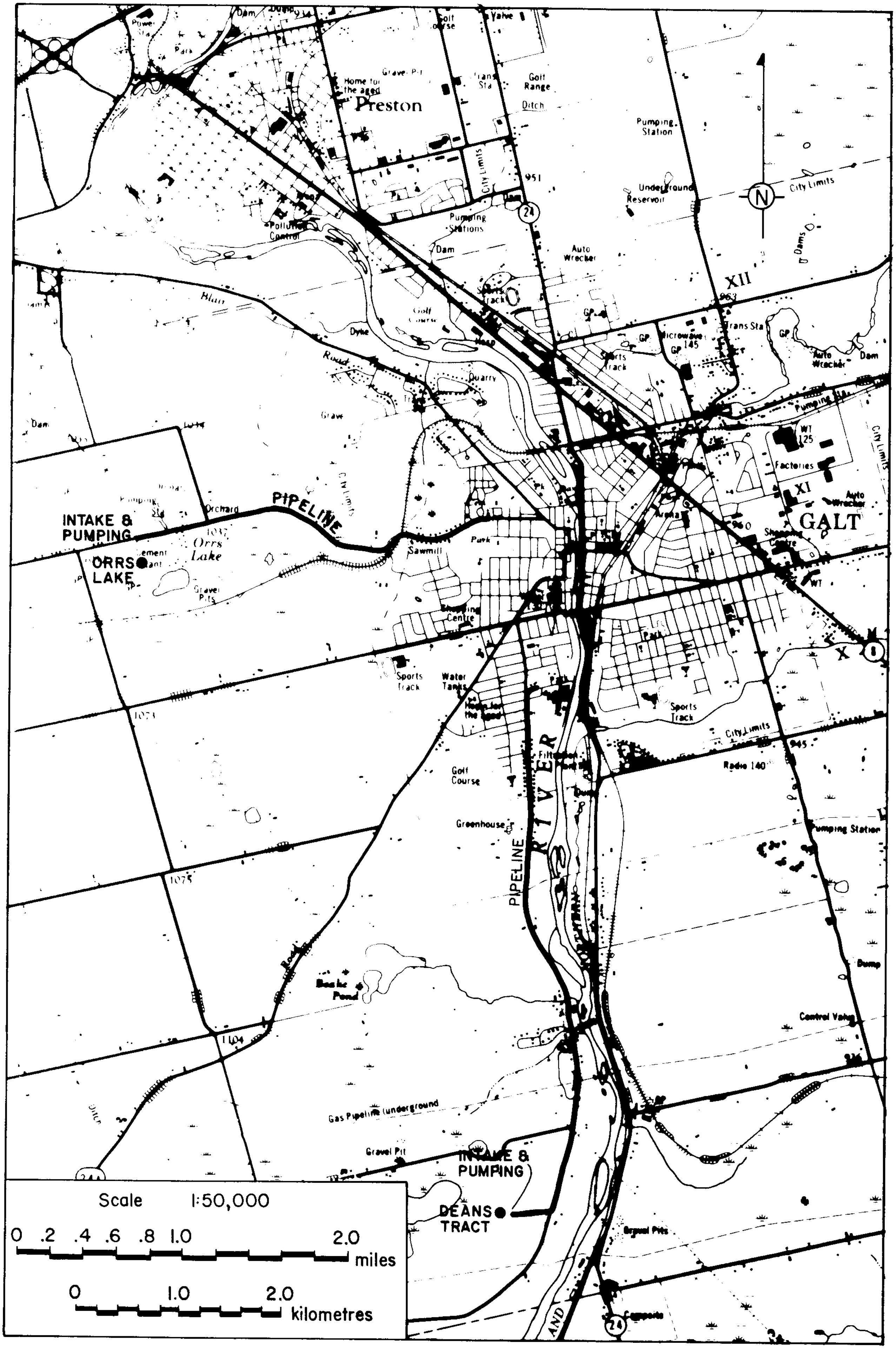


Figure D.1. Site of proposed ground water development projects near Cambridge.

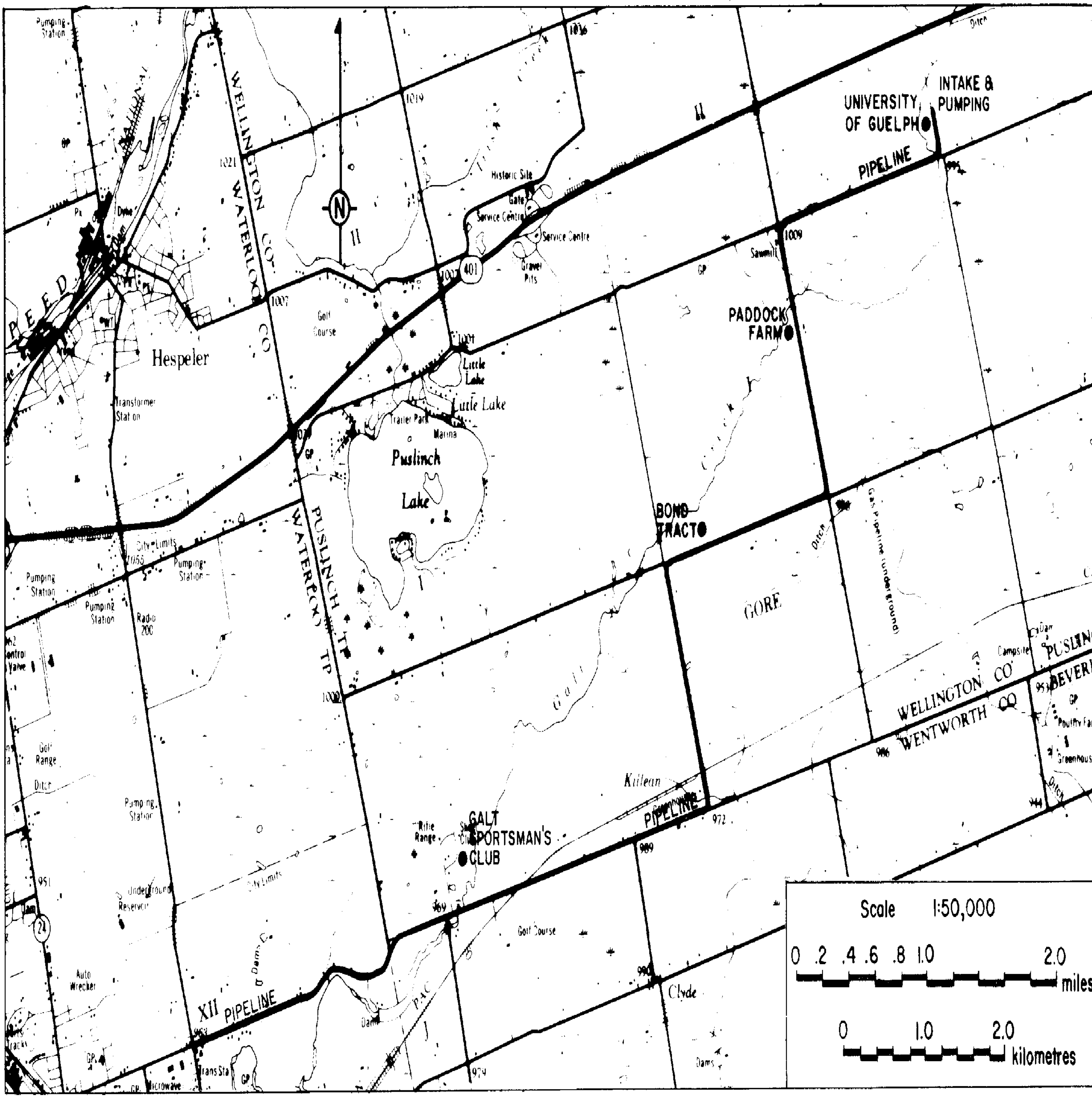


Figure D.2. Site of proposed ground-water development projects near Cambridge.

Connection to the distribution system will be by a 12-inch watermain, or larger if additional supplies are added, to a common main.

The proposed watermain layout is included in the figures D.1 and D.2.

C. Costs (1979)

(i) Capital

(a) Exploration		$225,000
(b) Land - 6 sites @ $2,500/site		$ 15,000
(c) Well construction		
- Drilling: 4,250 gpm @ $95/gpm	$403,750	
- Pump-houses:		
- 6 @ $10,000/house	$ 60,000	
- Hydro: 10 miles @ $20,000/mile	$200,000	
- Mechanical & electrical: 6 @ $30,000	$180,000	
- Supervisory controls: 6 @ $5,000	$ 30,000	
	$873,750	$873,750
(d) Feeder Mains		
- 37,600 lin. ft. 12" Ø @ $30/Ft.	$1,128,000	
- 33,600 lin. ft. 18" Ø @ $50/ft.	$1,680,000	
- 11,200 lin. ft. 24" Ø @ $75/ft.	$ 840,000	
	$3,648,000	$3,648,000
Total Capital Cost		$4,761,750

(ii) Annual Operation & Maintenance

(a) Electric Power	
6 @ $1,300/yr.	$ 7,800
(b) Physical & Chemical Rehabilitation	
6 @ $1,000/yr.	$ 6,000
(c) Pump & Motor Maintenance	
6 @ $700/yr	$ 4,200
(d) Labour	
1/2 man year @ $20,000/yr.	$10,000
(e) Transportation	$ 2,000
Total Annual Operation & Maintenance	$30,000

APPENDIX E

INDUCED INFILTRATION PROJECTS FOR KITCHENER-WATERLOO

INDUCED INFILTRATION PROJECTS FOR KITCHENER-WATERLOO

A. Description

Exploration to determine the location of river-connected aquifers began in 1975, with funding provided by the Province of Ontario to a maximum of $75,000. The program was completed in 1976 at a total cost of approximately $110,000.

Four satisfactory locations were found as identified in Figure E.1. These are:

(i) Forwell
(ii) Pompeii
(iii) Woolner
(iv) Breslau

Fortunately, these sites are adjacent to the City of Kitchener where access to the distribution system and electrical power are readily available.

Development of the sites for water supplies has begun. As of September 1979, three (3) collector wells have been constructed on the Woolner site, two (2) on the Forwell site (in operation), and four (4) small wells on the Pompeii site. Construction on the Breslau site will commence upon development of an industrial subdivision nearby. This has tentatively been scheduled for 1983.

The results of hydrogeologic investigations and test pumping of existing collector wells indicate the availability of the following water quantities:

Woolner	2.448 (mgd)
Forwell & Pompeii	2.448 (mgd)
Breslau	1.008 (mgd)
	5.904 (mgd)

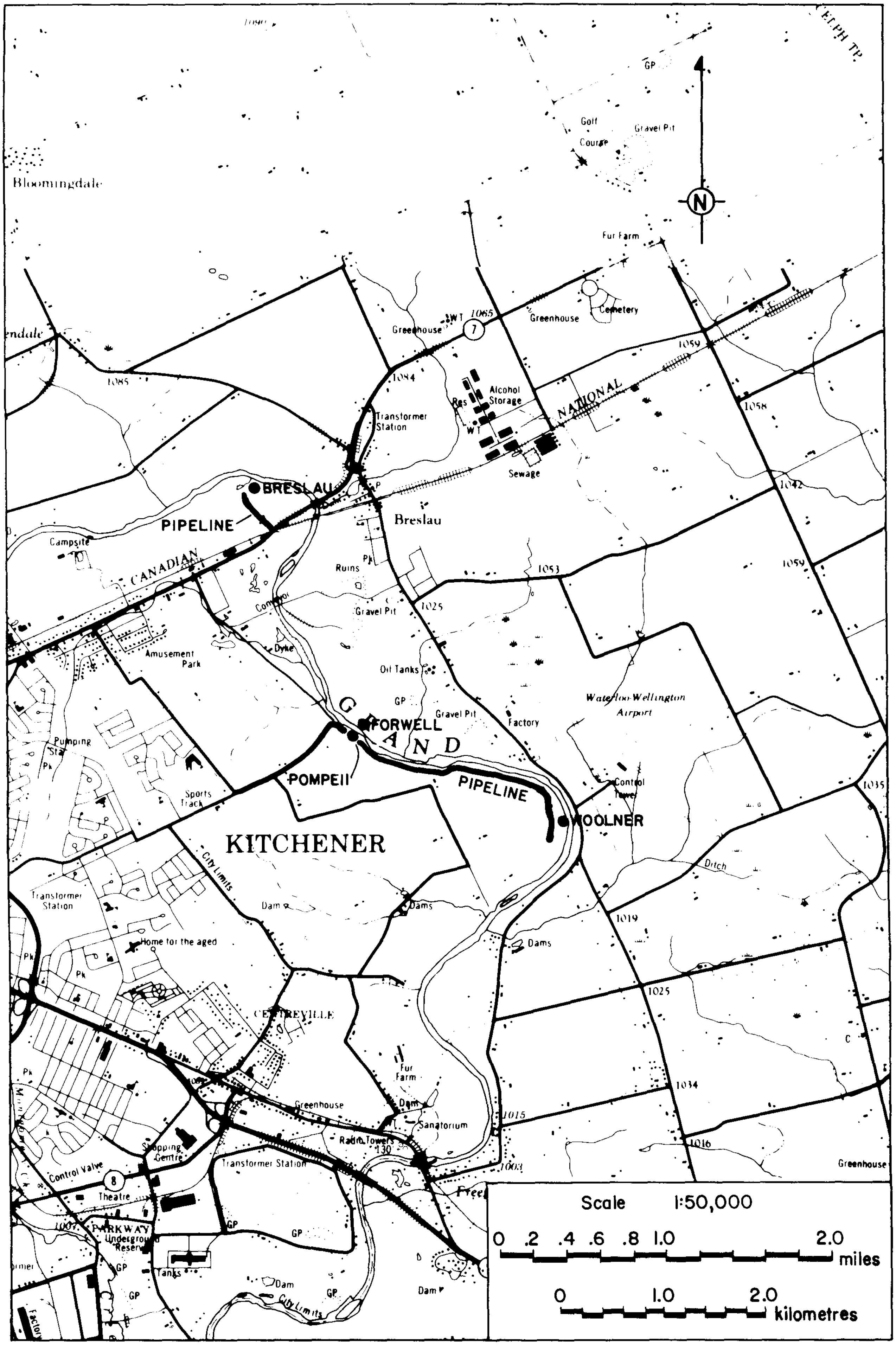


Figure E.1. Site of existing and proposed induced infiltration projects for Kitchener-Waterloo.

Following complete development of these sites and an accumulation of pumping history, it is felt that water production can be improved by the scouring of the river bottom, the construction of infiltration channels and an increase in river levels by the construction of weirs.

B. <u>Design Criteria</u>

The collector wells are constructed utilizing an open-cut method. Stainless steel well screen, 100 feet in length, is welded to a 24-inch vertical riser at midpoint and two (2) vertical clean-out risers at the ends. This assembly is then lowered into the excavation with the screen supported and surrounded by select material. Backfilling of the remainder of the trench is carried out with native material.

In order to ensure that flood damage is precluded, the pump, motor and associated mechanical and electrical equipment is constructed on a filled area above the Regional flood line. The pump house is simply a prefabricated, prepainted, steel building supported on a flat concrete slab.

One of the extraordinary costs associated with flood plain construction is the provision of electric power. It has been found that power distribution below the Regional flood line must be accomplished with buried submarine cable.

The water production equipment data are as follows:

Screens	-	100 lineal feet of 8-inch diamter stainless steel
Risers	-	24-inch diameter steel casing
Clean-outs	-	8-inch diameter steel casing
Pumps	-	Vertical turbine
	-	700 gpm @ 300 foot head (approx.)

Motors	-	100-125 H.P.
Chlorination	-	Chemical feed pump utilizing sodium hypochlorite as disinfectant
Mechanical & Electrical	-	Check valve (APCO silent check)
	-	Turbine water meter
	-	Air relief valve
	-	Remote start-stop and status to central control
	-	Reduced voltage starter
	-	15 KW heater

The well pumps are normally connected to the trunk transmission main with a 10 or 12-inch diameter connection.

C. Costs (1979)

(i) Capital

(a)	Exploration					$136,000
(b)	Land	@ $4000/ac.				
		@ $2000/ac. easement				$120,000
(c)	Well Construction -	Excavation		$20,000		
		Backfill		$10,000		
		Materials		$35,000		
		Test Pumping		$10,000		
			7 @	$75,000	=	$525,000
(d)	Pump House		7 @	$10,000	=	$ 70,000

(e)	Hydro (550 v.) @ $20,000/mi.	- Forwell	$110,000		
		Pompeii	$ 20,000		
		Woolner	$150,000		
		Breslau	$ 50,000		
			$ 330,000	=	$ 330,000
(f)	Mechanical & Electrical	7 @	$ 30,000	=	$ 210,000
(g)	Connecting Mains				
	1,700 lin. ft. - 30" Ø @ $90/ft	=	$ 153,000		
	11,600 lin. ft. - 24" Ø @ $75/ft	=	$ 870,000		
	1,600 lin. ft. - 18" Ø @ $50/ft	=	$ 80,000		
	800 lin. ft. - 12" Ø @ $30/ft	=	$ 24,000		
			$1,127,000	=	$1,127,000
	Total Capital Costs				$2,518,000

Note:

To the end of 1979, a total of $1,186,438 had been expended on this program.

(ii) Annual Operation & Maintenance

(a)	Electric Power 7 wells @ $1,300/yr.	=	$ 9,100
(b)	Physical & Chemical Rehabilitation 7 @ $1,000/yr.	=	$ 7,000
(c)	Pump & Motor Maintenance 7 @ $700/yr.	=	$ 4,900
(d)	Chemical 7 @ $600/yr.	=	$ 4,200
(e)	Labour 1/2 man yr. @ $20,000/yr.	=	$10,000
(f)	Transportation	=	$ 700
	Total Annual Operation & Maintenance	=	$35,900

APPENDIX F

ARTIFICIAL GROUND-WATER
RECHARGE PROJECT NEAR MANNHEIM

ARTIFICIAL GROUND-WATER RECHARGE PROJECT NEAR MANNHEIM

A. Description

A detailed investigation of the feasibility of artificial recharge of ground water to supplement water supplies in the Regional Municipality of Waterloo was completed by the Province of Ontario in 1974. Two sites were recommended for further investigations:

(i) Roseville
(ii) Mannheim

The immediate use of the Roseville site was precluded because the scheme depended on the construction of the proposed Ayr reservoir.

In 1976, the Regional Municipality of Waterloo initiated artificial recharge studies in the Mannheim area. Unfortunately, however, access to the proposed recharge area was denied by the property owner and the study was discontinued until July 1979, when the property was purchased by the Regional Municipality of Waterloo.

In the interim, geologic data were compiled from domestic and municipal well logs in the area and a preliminary mathematical model was constructed and tested. This model will be improved with the addition of geologic information as it becomes available during future studies at this site.

It is estimated, conservatively, that water quantities in the order of 20 mgd can be recharged and recovered at the site. Initially, the recharge will be operated for approximately 3 months in the spring each year when flows in the Grand River are high. The ultimate operation of the scheme will likely have to be carried out throughout the year to reach the 20 mgd objective.

B. Design Criteria

It is proposed that raw water from the Grand River be pumped to the recharge site from a point immediately upstream of the Kitchener Waste Water Treatment Plant (Figure F.1). Until the dynamics of recharge and recovery are known, the capacity of the raw water pumping station and the pipeline sizing must be assumed.

For purposes of costing, it is assumed that 20 recovery wells of 1 mgd each will be required and connected to an expanded Mannheim reservoir through a network of 12, 18 and 24-inch water mains.

It is also assumed that no pretreatment of the water will be carried out at the raw water pumping station. Pretreatment, if required, will be performed at the recharge site.

While costs are outlined below in 1979 dollars, it should be realized that the construction of recharge ponds and recovery wells will be performed on a staged basis as required.

C. Costs (1979)

(i) Capital

(a) Intake and Pumping Station at the Grand River

Weir		$ 100,000
Intake structure		150,000
Pumping station structure		200,000
Pumping station electrical		100,000
Pumping station mechanical		100,000
Pumping station controls		50,000
Sand for pumping station		20,000
	Sub-total	720,000

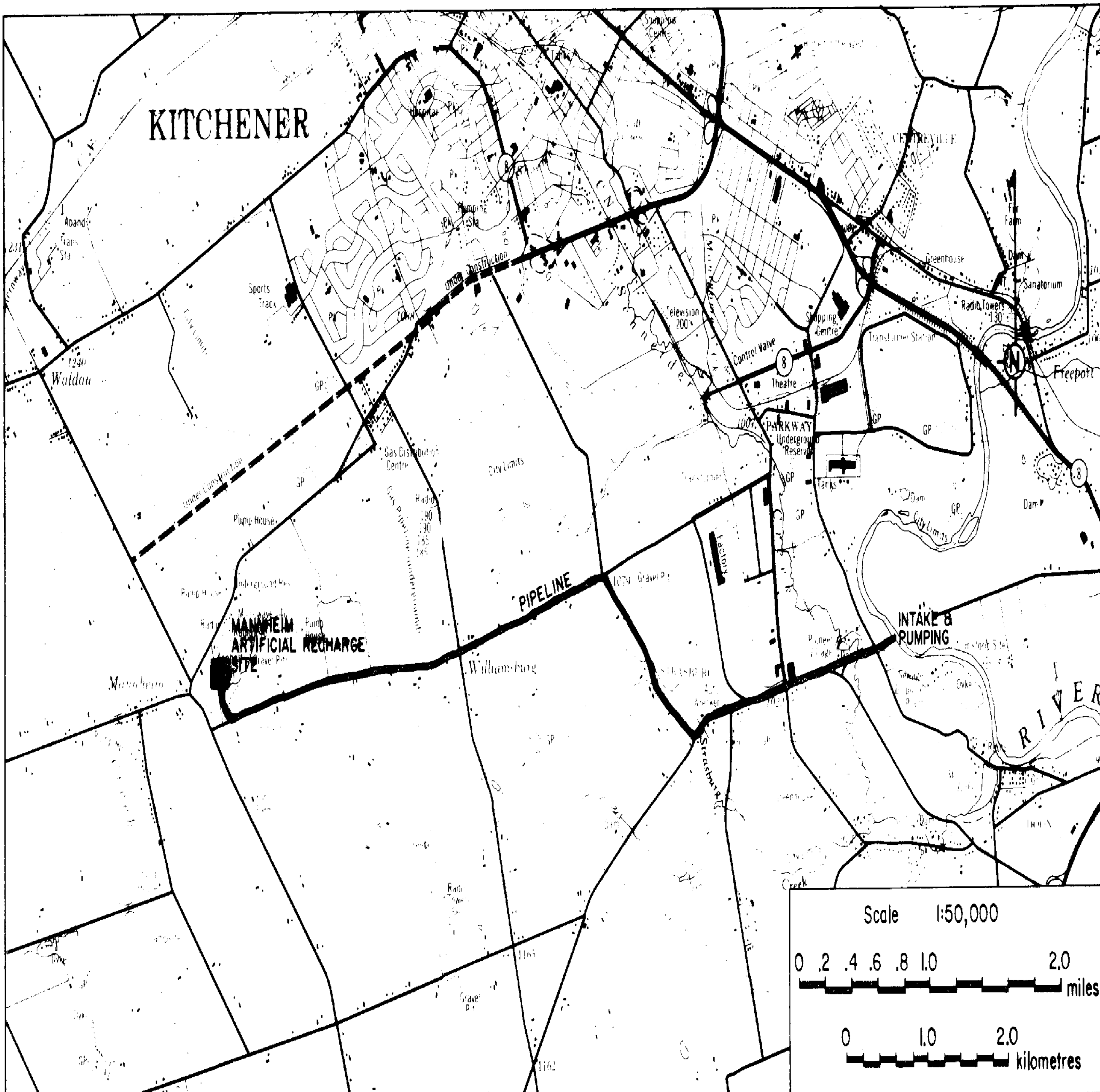


Figure F.1. Site of proposed artificial recharge project near Mannheim.

(b) Pipeline

30,000 ft. of 18" Ø watermain at $47/ft.*	1,410,000
30,000 ft. of 42" Ø watermain at $110/ft*	3,300,000
Sub-total	4,710,000

(c) Recharge Site

Land area	700,000
Site preparation - excavation and grading	300,000
on-site distribution	100,000
monitoring equipment	50,000
pre-treatment equipment	25,000
site, office and storage	75,000
Recovery wells**-20 at $135,000 each	2,700,000
Collection system**	
-1,300 ft. of 12" Ø pipe at $30/ft.	39,000
-7,600 ft. of 18" Ø pipe at $40/ft.	380,000
-3,800 ft. of 24" Ø pipe at $75/ft.	285,000
Sub-total	4,654,000

(d) Miscellaneous

Exploration and Pilot Project	$ 220,000
Additional storage at Mannheim at $0.234/gal.	2,340,000
Sub-total	2,560,000

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* installed costs

** recovery wells and collection system to be installed on a staged basis

(e) Capital Costs Summary

(i) Intake and pumping station		720,000
(ii) Pipeline		4,710,000
(iii) Recharge site		4,654,000
(iv) Miscellaneous		2,560,000
	Sub-total	12,644,000
	Engineering and Contingency (20%)	2,528,800
Total Capital Costs		15,172,800

(ii) Annual Operation & Maintenance

(a)	Electrical power	
	20 wells at $1,300/year/well	26,000
(b)	Physical and Chemical Well Rehabilitation	
	20 wells at $1,000/year/well	20,000
(c)	Pump and Motor Maintenance	
	20 wells at $700/year/well	14,000
(d)	Chemical	
	20 wells at $600/year/well	12,000
(e)	Labour	
	2 man years at $20,000 each	40,000
(f)	Transportation	4,000
(g)	Scarify recharge ponds	10,000
	Total Annual Operation and Maintenance	126,000

APPENDIX G

ARTIFICIAL GROUND-WATER RECHARGE PROJECT NEAR ROSEVILLE

ARTIFICAL GROUND-WATER RECHARGE PROJECT NEAR ROSEVILLE

A. Description

This project involves artificially recharging ground water in the vicinity of Roseville (UTM grid reference 425986 - 1:50,000 map sheet 40P/8), approximately 5.6 miles south of Kitchener, Ontario (Figure G.1).

The proposed development consists of a raw water supply facility (Ayr reservoir) with a micro-straining and chemical treatment plant and a pumping station, a series of infiltration basins, seventy-five (75) recovery wells, and a storage reservoir and pumping station for the recovered ground water (Figure G.2) (Hydrology Consultants Limited, 1974).

The recommended recharge area is within a kame-esker complex consisting of a relatively thick (20-100 ft) sequence of surficial outwash sands and gravels that are underlain by interbedded silt, clay and glacial till. The permeability of the sands and gravels in the recharge area was estimated to be approximately 10^{-2} cm/sec. This proposed development is expected to yield at least about 20 mgd, allowing seasonal recovery of up to about 35 mgd.

B. Design Criteria

The raw water supply intake facility consists of a mechanical screen, a micro-straining plant rated at 20 mgd, a treatment plant for the micro-strainer's waste water, a pumping station rated at 20 mgd, and three miles of 42-inch diameter pipeline to deliver the raw water to the recharge site.

The recharge facility consists of infiltration basins and a network of raw water distribution pipes. It is estimated that about 20 mgd could be recharged through strategically located basins with a combined surface area of about 9 acres.

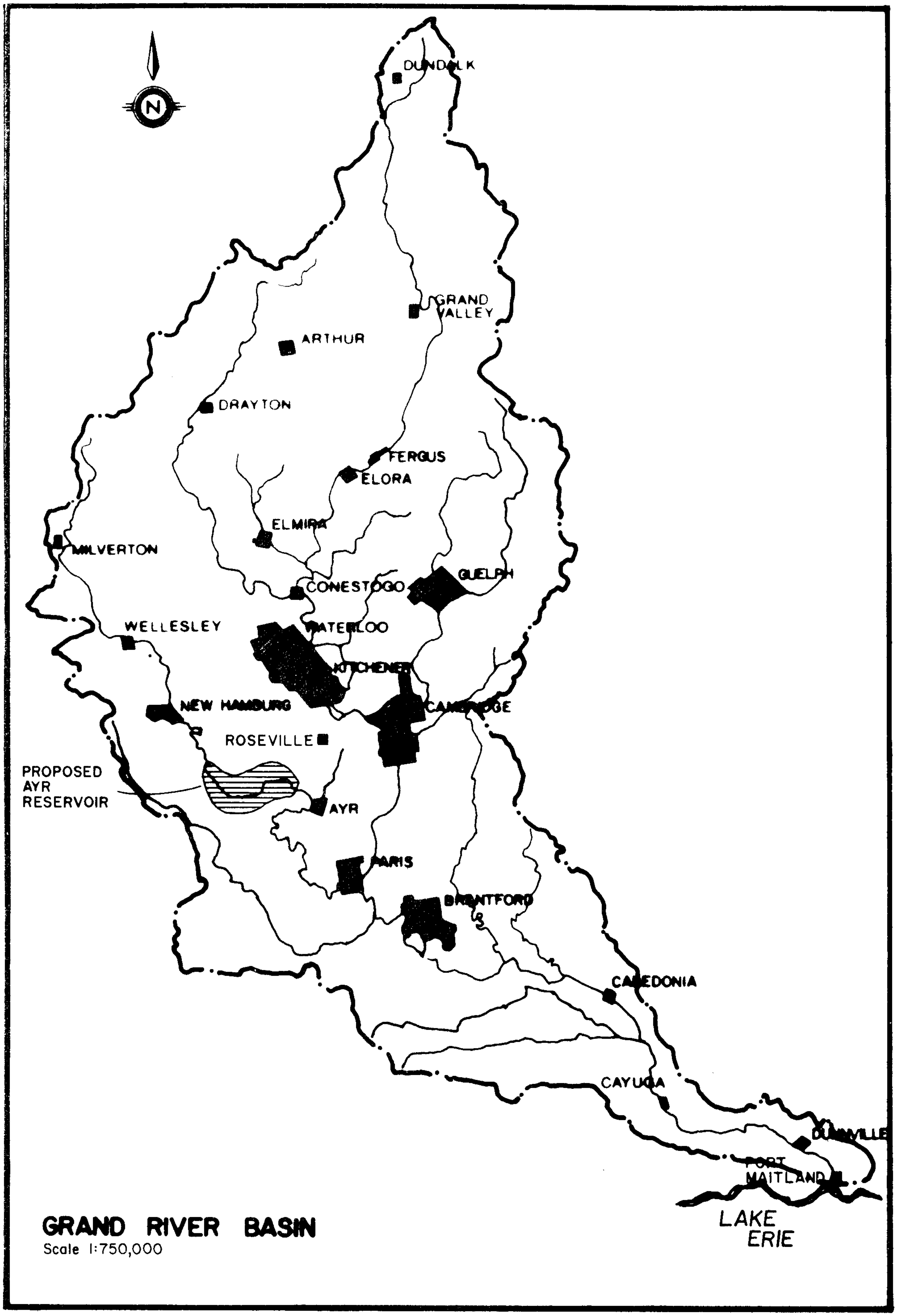

Figure G.I. Site of proposed artificial ground-water recharge project near Roseville.

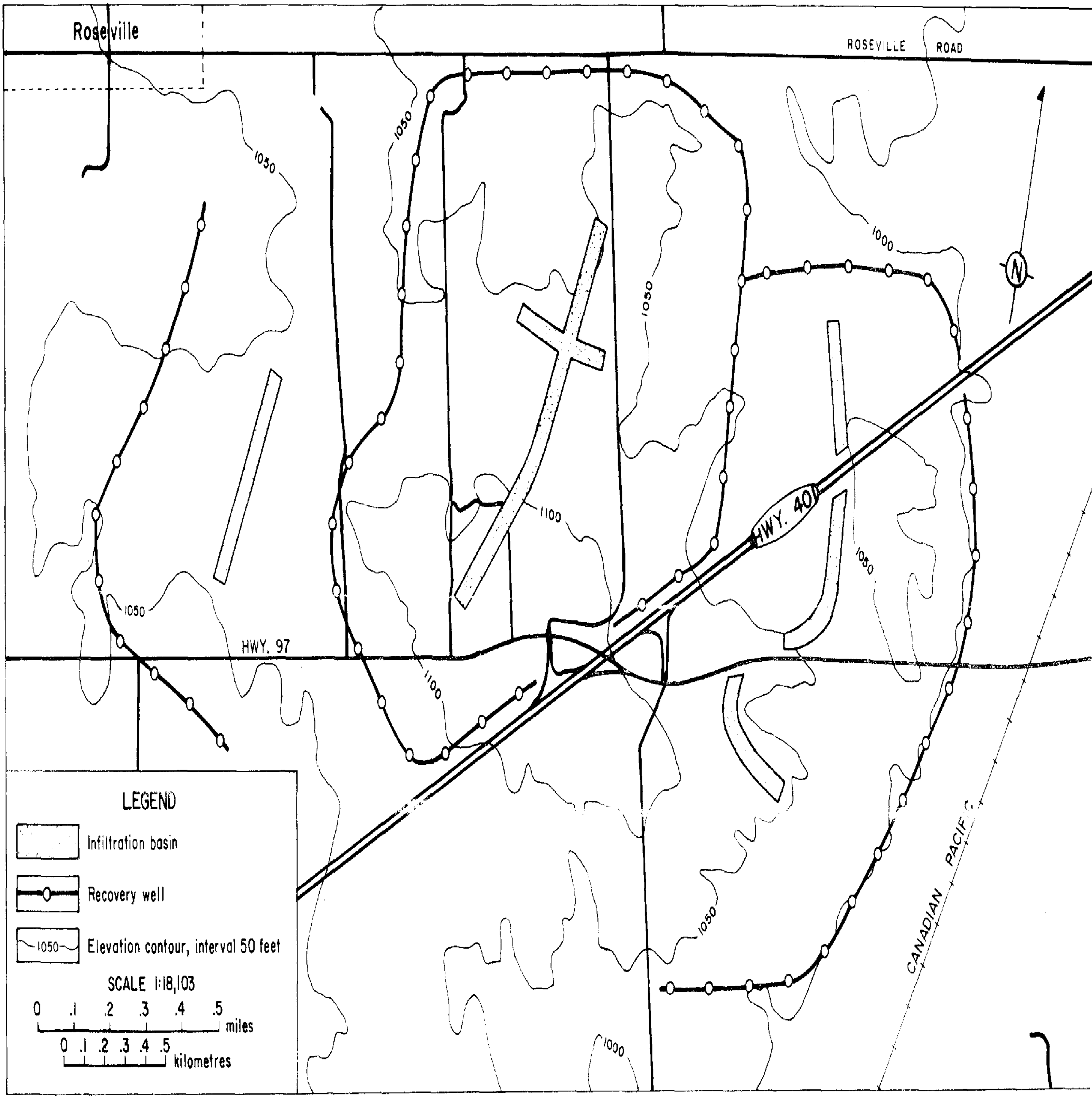

Figure G.2. Site of proposed artificial recharge project near Roseville.

The (oversized) recovery facility consists of 75 recovery wells, each rated at 0.5 mgd, a 5 million gallon water storage reservoir, a pumping station rated at 35 mgd, and five miles of 54-inch diameter pipeline to deliver the recovered ground water to the Kitchener-Waterloo distribution system.

C. Costs (1979)

The 1979 capital costs of this project are those listed by Hydrology Consultants Limited (1974), up-dated to 1979 dollar values. The up-dating was based on the following formula:

$$\frac{211.4}{145.0} \times 1974\ \$ = 1979\ \$$$

where 211.4 = 1979 construction cost index

145.0 = 1974 construction cost index

The construction cost indices are composite material and wage costs in Ontario, compiled by Southam Business Publications Limited. The 1979 index is an average composite index for the first six months of 1979.

The 1979 costs for the main pipelines (raw water and recovered ground water) were not determined by the up-dating procedure outlined above. Due to the "specialty" nature of large-diameter pipe, actual 1979 costs of this item were obtained from Standard Pressure Pipe Company (Toronto). The 42-inch raw water pipe costs $48 per foot. A 56-inch diameter pipe is no longer available as a standard item and thus the recovered ground-water pipeline is costed as being 54 inches in diameter ($92 per foot). These costs are for concrete pipe.

(i) Capital (excluding land costs):

(a) Raw Water Supply

Intake with mechanical screen	$ 558,300	
Micro-straining plant, 20 mgd	$ 583,000	
Treatment plant for waste water from micro-strainers, estimated	$ 291,600	
Pumping station, 20 mgd	$ 218,700	
Main pipeline 0 42", 3 miles	$ 768,200	$ 2,418,800

(b) Infiltration Plant

Raw water distribution pipes	$ 1,151,800	
Infiltration basins, 400,000 sq. ft.	$ 437,400	$ 1,589,200

(c) Well Field

Wells with appurtenances 75 total $ 1,749,500		
Connecting pipelines	$ 3,829,500	$ 5,578,000

(d) Well Water Distribution System

Reservoir 5 mg	$ 729,000	
Pumping station 35 mgd	$ 379,100	
Main pipeline 0 54", 5 miles	$ 2,428,800	$ 3,536,900

(e) Design, Supervision $ 605,000

Total Capital Costs $13,221,000

ii) Annual Operation & Maintenance

Operation and maintenance costs are estimated to be about 14¢ per 1000 gallons of water distributed. This estimate is based on actual 1978 costs in the Regional Municipality of Waterloo. This does not include the cost of chlorination or debt retirement of capital costs.

APPENDIX H

NATURAL GROUND-WATER DEVELOPMENT PROJECT NEAR ROSEVILLE

NATURAL GROUND-WATER DEVELOPMENT PROJECT NEAR ROSEVILLE

A. Description

This project involves the development of ground water in the vicinity of Roseville (UTM grid reference 425986 - 1:50,000 map sheet 40P/8), approximately 5.6 miles south of Kitchener, Ontario (Figure H.1).

The proposed development consists of a number of municipal wells, a storage reservoir, a chemical treatment facility and a pumping station, all within an area of approximately ten square miles (Figure H-2).

The water-bearing formation in this target area, which was identified in well logs on file with MOE and confirmed in the field by test-drilling, consists of medium sand grading to medium gravel, and varies in thickness from about 10 to 30 feet. The average depth of this formation is about 110 feet.

Existing wells finished in this formation indicate high specific capacities, with adjusted theoretical yields ranging from 13 to 512 gallons per minute.

Based strictly on mathematical calculations, and assuming certain values necessary for the calculations, the total ground-water recharge from precipitation in this area is estimated to be approximately 5.5 million gallons per day.

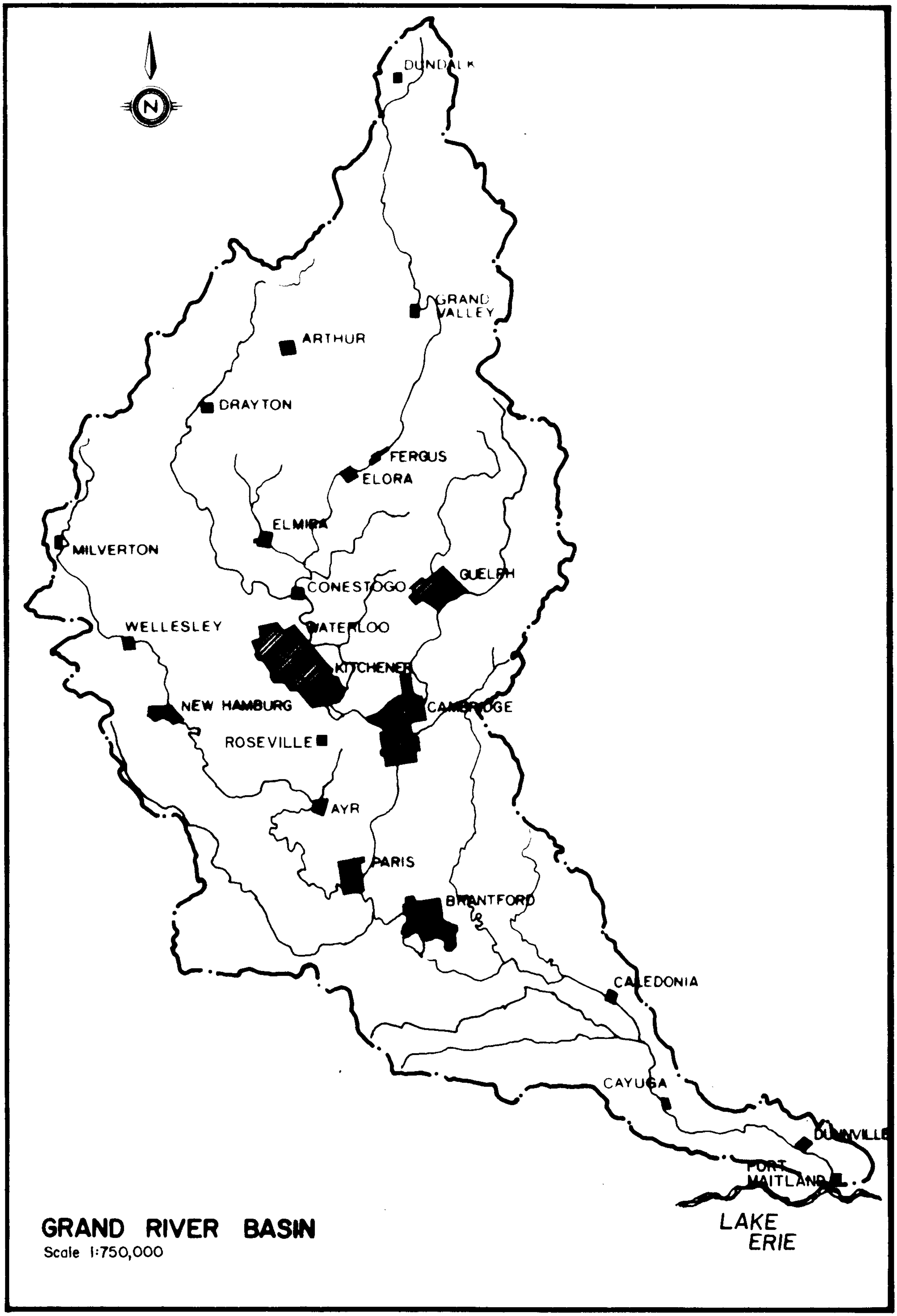

Figure H. I. Site of proposed natural ground-water development project near Roseville.

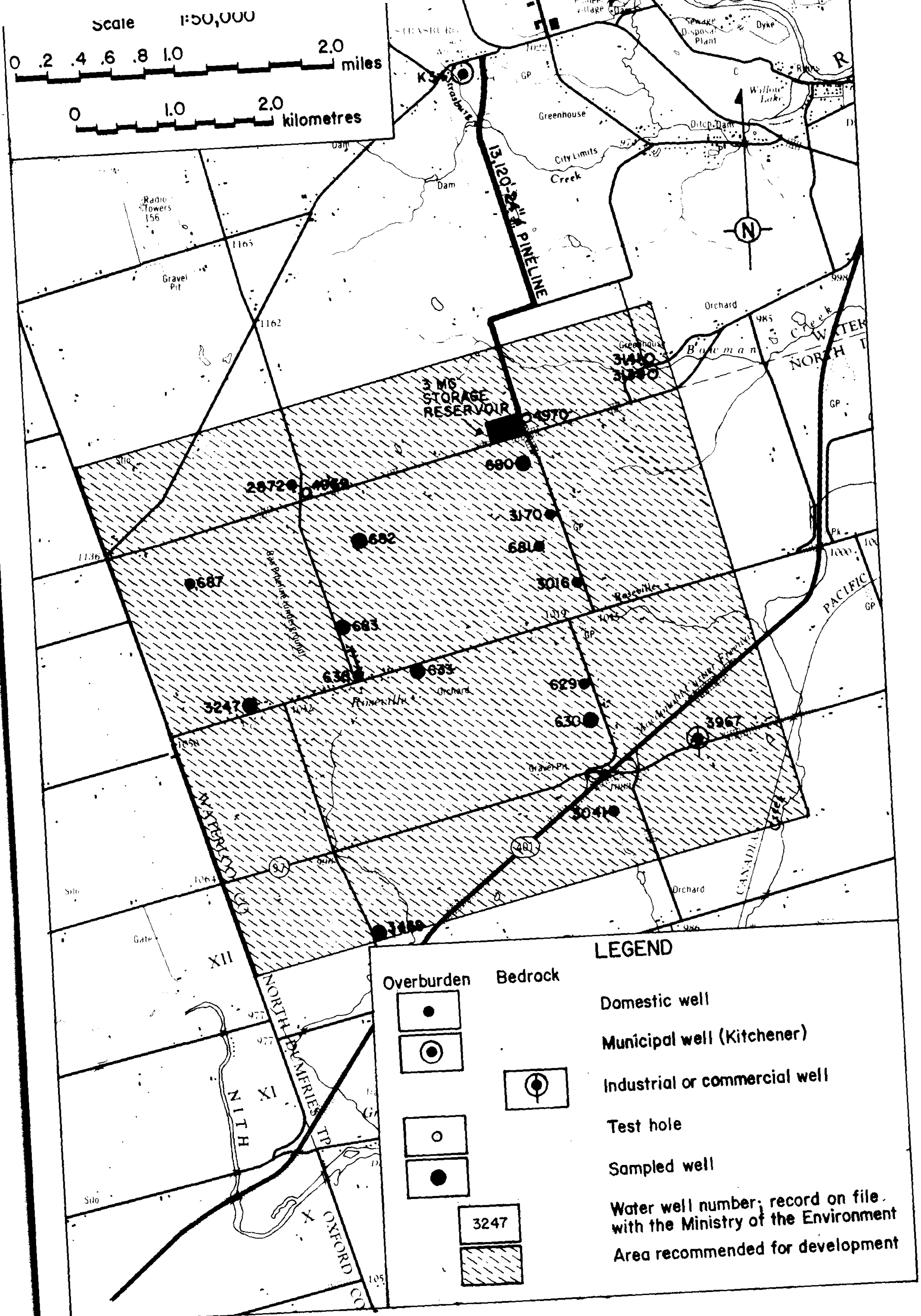

Figure H. 2. Recommended Roseville development area.

B. Design Criteria

The total recharge to the aquifer recommended for development was based on the following formula:

$$Q = 27.9 \, lhA$$

where l = leakance of aquitard
h = lowering of head across aquitard
A = surface area of aquifer.

The following estimates were made from available data:

$l = 10^{-3}$ gpd/ft^3
h = 40 feet
A = 10 miles^2.

In the above estimates, h and A are based on field data but l is an estimate obtained from the 1973 IWS report 'Kitchener-Waterloo, Groundwater Evaluation'. This leakance of 10^{-3} gpd/ft^3 appears to be too high, yielding a total infiltration rate of 11 mgd over the 10 square miles. Adjusting the leakance to 5×10^{-4} gpd/ft^3 would yield a total infiltration of about 5.5 mgd, which is a more realistic value.

Assuming a recovery potential of about 50 percent of the estimated total recharge rate (5.5 mgd), this formation might be developed to ultimately yield about 2-3 mgd for municipal purposes. It is estimated that each well in the formation should yield about 500 gpm.

A 3-million gallon storage reservoir might be required at the well field, along with a chemical treatment facility rated at 500 gpm for chlorination, and the addition of sodium silicate for iron removal. A pumping station would also be required to deliver approximately 3 mgd from the well field to the existing Kitchener-Waterloo distribution system. The closest point of entry into the existing system is at well K34. The pumping distance is about 2.5 miles with a head of 150 feet.

C. Costs (1979)

(i) Capital (excluding land costs):

(a) Exploration

$30 per foot
110 feet per test-hole - $3,300.
5-10 test-holes per 1 production well - $16,500-$33,000
4 wells $66,000-$132,000

(b) Well Construction and Development

$95 per gallon per minute
500 gpm per well- $47,500
4 wells $ 190,000

(c) Pump House

$10,000 per well
4 wells $ 40,000

(d) Well Pumps and Related Controls

$ 35,000 per well
4 wells $ 140,000

(e) Storage Reservoir

22¢ per gallon
3 million gallons $ 660,000

(f) Chemical Treatment

chlorination $ 5,000
sodium silicate $ 5,000

Sub-total 1,106,000 - 1,172,000

(g) Pumping Station

superstructure		$200,000
mechanical		$100,000
electrical		$100,000
pumps - 1.5 mgd (3)	- $15,000 each	$ 45,000
deisel generating station (standby)		$ 60,000
		$505,000

(h) Watermain *

12-inch diameter	- $30 per foot	
	- 24,000 feet	$720,000
16-inch diameter	- $45 per foot	
	- 1,000 feet	$145,000
18-inch diameter	- $50 per foot	
	- 6,200 feet	$310,000
24-inch diameter	- $75 per foot	
	- 13,120 feet	$984,000
		$2,159,000

Total Capital Costs: $3,770,000 - $3,836,000

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* NOTE: The 24-inch diameter watermain connects the reservoir to the existing Kitchener-Waterloo distribution system at well K34, as shown in Figure H.2. The 12 to 18-inch watermains connect the individual wells to the storage reservoir. The costs of these smaller diameter watermains are based on a theoretical distribution of wells, such that the four drawdown cones (assuming a maximum diameter of one mile) interfere as little as possible with the existing wells shown in Figure H.2.

This theoretical distribution of wells was carried out solely for determining the approximate costs of watermains within the development area. The final location of wells will depend on detailed hydrogeologic investigations to be carried out prior to development of the area.

(ii) Annual Operation & Maintenance

Operation and maintenance costs are estimated to be about 14¢ per 1000 gallons of water distributed. This estimate is based on actual 1978 water costs in the Regional Municipality of Waterloo. This does not include the cost of chlorination or debt retirement of capital costs.

Specific Capacities and Adjusted Theoretical Yields of Wells in the Recommended Roseville Development Area

Well Number	Surface Elevation (ft below ground)	Water Found (ft below ground)	Static Level (ft below ground)	Pumping Level (ft below ground)	Drawdown (ft)	Test Pumping Rate (gpm)	Specific Capacity (gpm/ft)	Available Head (ft)	Adjusted Theoretical Yield (gpm)
629	1070	101	46	48	2	15	7.50	55	206.25
630	1070	90	40	46	6	20	3.33	50	83.25
633	1055	159	39	48	9	5	0.55	120	33.33
636	1050	60	40	50	10	13	1.30	20	13.00
680	1135	170	70	83	13	22	1.69	100	84.50
681	1025	72	55	60	5	20	4.00	17	34.00
682	1075	92	36	80	44	20	0.45	56	12.73
683	1025	218	80	100	20	15	0.75	138	51.75
687	1105	114	67	76	9	12	1.33	47	31.25
2872	1125	104	75	80	5	15	3.00	29	43.50
3016	1075	80	50	55	5	15	3.00	30	45.00
3139*	1000	116	FLW	11	11+	87	7.91	116	458.78
3141	1040	88	38	48	10	205	20.50	50	512.50
3170	1100	94	55	70	15	15	1.00	39	19.50
3247	1030	45	27	28	1	12	12.00	18	108.00
3401*	1040	249	85	86	1	15	15.00	164	1230.00
3469	1000	52	18	20	2	20	10.00	34	170.00
3967**	1050	315	50	75	25	25	1.00	265	88.33

* Target formation present but not developed

** Target formation not present

APPENDIX I

MONTHLY MEAN AND ANNUAL MEAN MINIMUM
STREAMFLOWS FOR 15 GAUGES IN THE GRAND
RIVER BASIN, 1970 - 1977

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
Grand R. near Marsville - Gauge A14													
1970	31.0	39.0	35.0	77.0	65.0	21.1	41.7	7.1	19.0	41.1	91.9	134.0	50.2
1971	95.0	93.5	113.0	190.0	18.2	11.8	34.7	6.1	4.8	4.1	5.3	15.4	75.4
1972	70.0	54.0	67.6	157.0	17.8	13.6	31.2	21.8	2.0	4.4	57.8	49.0	45.5
1973	70.0	45.0	43.0	98.0	53.3	27.8	37.2	28.4	4.7	3.5	32.0	52.0	41.2
1974	105.0	92.0	83.0	148.0	85.9	18.6	11.6	11.9	6.0	8.0	11.8	39.0	51.7
1975	37.0	66.0	207.0	220.0	15.3	11.1	21.2	14.6	23.3	18.3	25.6	76.0	61.3
1976	50.0	54.0	561.	90.0	39.6	9.0	19.5	47.4	27.5	65.6	109.0	48.0	93.4
1977	44.0	45.2	57.0	54.6	10.5	4.8	11.5	28.2	37.8	115.0	85.9	102.0	49.7
Average Minimum Flow for Period of Record - 58.6 cfs													
Conestogo R. near Drayton - Gauge A39													
1970	12.6	13.9	11.2	56.8	19.0	5.0	4.1	1.5	2.0	6.1	26.3	105.0	21.9
1971	22.7	21.0	67.0	122.0	7.2	2.5	1.5	1.5	1.5	1.3	3.2	7.0	21.5
1972	26.0	24.5	26.0	230.0	5.6	4.5	2.0	1.0	.9	3.7	40.0	38.0	33.5
1973	25.0	19.5	18.4	18.8	22.9	7.9	.4	.3	.2	.4	7.4	8.0	10.8
1974	19.0	11.9	18.5	55.9	27.8	4.2	.9	.4	.7	2.0	3.3	8.8	12..8
1975	8.8	8.0	60.0	87.9	7.3	4.0	1.6	2.4	8.3	8.8	15.1	23.0	19.6
1976	13.0	20.8	107.0	33.8	12.4	4.0	6.7	3.5	3.5	10.5	18.4	13.8	20.6
Average Minimum Flow for Period of Record - 20.1 cfs													
Irvine Cr. near Salem - Gauge "Salem"													
1975	5.0	9.0	82.0	82.0	5.0	3.0	2.0	1.0	9.0	5.0	7.0	14.0	11.8
1976	9.0	22.0	158.0	28.0	9.0	3.0	5.0	3.0	3.0	7.0	11.0	12.0	22.5
1977	7.0	7.0	12.0	17.0	2.0	2.0	1.0	2.0	3.0	14.0	12.0	51.0	10.8
Average Minimum Flow for Period of Record - 15.0 cfs													

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual

Speed R. near Armstrong Mills - Gauge A40

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1974	19.0	20.0	40.0	56.1	39.5	14.5	5.9	2.8	4.4	7.6	12.1	14.0	19.7
1975	14.6	11.0	42.0	70.0	16.1	8.2	4.7	2.5	8.7	9.5	15.5	30.0	19.4
1976	16.5	40.0	135.0	54.3	25.5	9.8	9.4	7.5	6.7	15.2	24.8	17.1	30.2

Average Miniumu Flow for Period of Record - 23.1 cfs

Eramosa R. above Guelph - Gauge A29

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	25.9	27.5	29.2	92.6	60.0	22.9	23.9	15.3	27.8	32.7	50.5	73.3	40.1
1971	46.9	47.7	62.6	104.0	37.7	34.2	21.3	16.7	24.6	20.5	26.7	30.3	39.4
1972	47.0	47.0	52.0	98.2	41.7	34.1	25.6	18.9	18.5	21.8	39.9	42.0	40.6
1973	48.0	45.7	51.9	97.4	66.5	38.2	21.8	13.8	10.2	21.3	36.6	35.0	40.5
1974	49.7	55.6	82.8	94.5	82.9	48.7	20.1	17.4	18.3	21.2	28.9	31.0	45.9
1975	36.6	35.0	68.0	106.0	41.0	33.8	16.6	14.0	27.7	25.2	31.8	44.0	40.0
1976	33.0	45.5	179.0	112.0	81.0	43.3	29.5	23.5	25.2	37.0	37.0	32.0	56.5
1977	20.0	19.7	28.0	89.5	34.9	21.9	13.9	22.2	24.3	62.3	59.8	98.0	41.2

Average Minimum Flow for Period of Record - 43.0 cfs

Nith R. above Nithburg - Gauge A38

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1972	-	-	15.0	63.5	7.3	2.9	3.4	1.1	1.6	3.2	52.0	35.0	18.5
1973	20.0	10.6	10.4	16.2	19.8	3.5	1.3	1.5	1.1	1.5	5.8	13.0	8.7
1974	16.0	15.7	19.2	40.9	19.0	5.7	2.5	1.4	1.7	2.5	3.3	6.0	11.2
1975	6.9	6.5	41.0	52.5	5.3	4.2	2.0	1.6	21.1	12.2	16.6	27.1	16.4
1976	27.0	41.0	105.0	29.2	13.5	4.1	3.3	2.7	2.7	4.5	10.4	11.5	22.2

Average Minimum Flow for Period of Record - 15.4 cfs

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual

Whiteman's Cr. near Mr. Vernon - Gauge B8

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1972	47.0	40.5	43.0	158.0	68.2	48.1	34.0	29.7	32.5	40.6	145.0	111.0	66.5
1973	90.7	62.0	62.0	145.0	100.0	48.3	15.3	14.8	10.6	18.4	66.8	36.0	55.8
1974	48.0	47.1	150.0	132.0	105.0	58.0	39.7	23.6	23.9	28.1	32.5	53.1	61.8
1975	35.3	59.3	117.0	138.0	60.7	47.8	27.3	20.3	47.0	52.5	53.8	89.0	62.3
1976	75.0	95.6	311.0	117.0	84.8	51.0	55.7	58.5	47.3	66.3	61.9	54.8	89.1
1977	41.0	41.0	76.0	120.0	44.7	37.6	24.2	26.5	32.1	78.8	73.0	115.0	59.2

Average Minimum Flow for Period of Record - 65.8 cfs

Fairchild Cr. near Brantford - Gauge B7

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	18.0	26.0	32.5	61.6	27.6	9.0	7.6	8.1	8.4	14.9	50.6	70.0	27.8
1971	40.5	40.0	240.0	56.3	19.4	11.5	7.4	6.1	7.0	6.5	6.0	8.0	37.4
1972	29.0	20.0	48.0	95.7	24.9	16.5	15.2	13.7	11.5	13.0	94.5	52.0	36.2
1973	43.0	33.0	32.0	69.5	47.7	18.4	12.7	11.4	9.5	12.2	27.2	33.5	29.2
1974	56.0	52.0	85.0	89.5	58.7	24.8	5.9	9.4	8.8	12.9	17.2	22.5	36.9
1975	35.0	31.0	85.0	94.5	19.2	16.2	11.2	10.1	25.2	18.4	33.7	54.1	36.1
1976	47.5	80.0	227.0	70.9	63.7	29.7	19.9	16.3	15.0	26.9	27.1	21.8	53.8

Average Minimum Flow for Period of Record - 36.8 cfs

McKenzie Cr. near Caledonia - Gauge B10

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	14.4	8.1	18.0	24.3	16.6	4.4	5.8	0	1.5	5.4	23.2	40.0	13.5
1971	10.5	10.0	75.1	25.2	10.7	1.1	.02	.9	5.1	5.4	3.5	5.0	12.7
1972	18.0	7.0	17.0	45.1	12.6	6.3	2.9	3.4	5.1	3.6	32.2	27.0	15.0
1973	20.0	14.0	13.9	27.7	31.6	7.5	5.7	3.2	2.4	3.3	18.4	16.3	13.7
1974	26.0	19.5	42.0	27.5	24.4	8.9	4.1	2.4	3.8	4.0	5.8	11.5	15.0
1975	10.0	12.5	42.0	35.0	8.6	10.4	4.3	5.0	9.6	10.9	12.6	25.0	15.5
1976	24.0	48.0	82.5	22.6	25.8	8.5	19.6	4.1	3.9	12.5	15.0	9.0	23.0

Average Minimum Flow for Period of Record - 15.5 cfs

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual

Grand R. at Doon - Gauge "Doon"

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	133.0	222.0	219.0	902.0	340.0	109.0	147.0	73.0	164.0	245.0	384.0	470.0	284.0
1972	246.0	208.0	291.0	950.0	161.0	168.0	124.0	88.0	80.0	120.0	412.0	214.0	255.2
1973	270.0	193.0	365.0	556.0	401.0	195.0	63.0	108.0	62.0	122.0	208.0	111.0	221.2
1974	306.0	132.0	365.0	660.0	549.0	275.0	139.0	111.0	112.0	134.0	157.0	167.0	258.9
1975	161.0	196.0	569.0	702.0	229.0	157.0	119.0	96.0	225.0	193.0	283.0	305.0	269.4
1976	-	-	180.10	657.0	334.0	133.0	130.0	155.0	97.0	217.0	270.0	-	421.3

Average Minimum Flow for Period of Record - 285 cfs

Grand R. at Galt - Gauge A3

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	239.0	377.0	366.0	1422.0	622.0	175.0	227.0	160.0	292.0	390.0	606.0	763.0	469.9
1971	359.0	322.0	778.0	1205.0	361.0	311.0	210.0	174.0	187.0	169.0	246.0	245.0	380.6
1972	436.0	360.0	479.0	1385.0	293.0	280.0	217.0	151.0	150.0	214.0	642.0	340.0	424.8
1973	470.0	346.0	581.0	835.0	645.0	307.0	101.0	167.0	107.0	202.0	359.0	192.0	359.3
1974	488.0	255.0	658.0	946.0	937.0	465.0	233.0	181.0	189.0	229.0	264.0	288.0	427.8
1975	259.0	326.0	787.0	961.0	408.0	224.0	197.0	157.0	352.0	312.0	460.0	438.0	410.9
1977	241.0	275.0	349.0	772.0	265.0	196.0	129.0	214.0	279.0	729.0	550.0	567.0	380.5

Average Minimum Flow for Period of Record - 408 cfs

Grand R. at Brantford - Gauge B1

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1970	449	656	611	2317	917	573	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1971	537	504	1543	1865	686	516	372	335	343	321	361	301	640.3
1972	650	631	909	2701	739	515	359	332	320	427	1233	767	798.6
1973	1220	624	966	1435	1196	571	314	357	268	364	583	597	707.9
1974	1041	566	1201	1582	1458	724	434	278	331	400	480	476	747.6
1975	506	597	1428	1854	764	431	303	263	521	478	564	745	704.5
1977	478	589	736	1444	432	348	274	380	463	1096	918	1126	690.3

Average Minimum Flow for Period of Record - 726 cfs

Apr. May June July Aug. Sept. Oct. Nov. Dec. Annual

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
Nith R. at New Hamburg - Gauge A18													
1970	29.0	33.3	30.7	142.0	47.9	21.1	16.4	13.1	14.7	34.9	75.7	109.0	47.32
1971	42.6	43.0	147.0	126.0	27.9	18.7	16.5	12.3	13.1	16.2	16.7	23.5	41.96
1972	53.8	34.6	39.0	127.0	20.3	20.4	18.0	14.8	13.6	19.8	96.5	59.0	43.10
1973	66.3	38.6	37.0	51.7	49.9	8.0	12.0	15.6	12.4	15.9	31.7	31.8	30.86
1974	42.0	37.5	48.3	100.0	51.9	26.5	16.6	10.1	9.8	13.6	21.1	20.0	33.12
1975	23.4	24.4	94.0	82.6	28.4	24.5	15.5	13.1	43.7	38.2	40.9	58.8	40.62
1976	45.0	71.8	232.0	69.1	42.9	19.3	17.9	16.6	15.9	23.9	33.1	27.0	51.21
Average Minimum Flow for Period of Record - 41.2 cfs													
Nith R. at Canning - Gauge A10													
1970	63.6	109	107	394	157	95.3	83.9	78.1	78.8	134	195	179	139.56
1971	161	165	330	281	117	89.6	74.8	63.5	71.9	72.6	83.4	108	134.82
1972	165	125	130	324	128	116	105	91.9	88.6	107	230	195	150.46
1973	190	161	160	233	193	124	89.7	88.8	74.0	87.1	127	130	129.80
1974	228	150	204	304	213	137	102	77	78.5	95.9	106	100	149.62
1975	105	160	261	274	113	113	79.9	80.3	156	129	144	240	154.60
1976	132	190	535	240	172	122	108	101	97.6	122	140	100	171.63
Average Minimum Flow for Period of Record - 147 cfs													
Speed R. below Guelph - Gauge A15													
1970	56.0	82.0	83.0	225.0	111.0	42.4	32.0	38.9	59.0	69.2	119.0	168.0	9.05
1972	110.0	89.6	100.0	229.0	84.3	73.6	52.1	37.6	37.0	45.4	108.0	88.9	88.0
1973	125.0	97.0	95.0	183.0	128.0	84.4	25.9	29.3	21.7	36.4	70.3	61.0	79.8
1974	103.0	110.0	151.0	176.0	167.0	89.1	43.4	26.6	27.8	36.1	44.2	53.0	85.6
Average Minimum Flow for Period of Record - 86.0 cfs													